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of  
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DETAILS  
OF  
STEEL HIGHWAY BRIDGES

BY  
JOHN A. CALLAN

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THESIS  
FOR THE  
DEGREE OF BACHELOR OF SCIENCE  
IN  
CIVIL ENGINEERING

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COLLEGE OF ENGINEERING  
UNIVERSITY OF ILLINOIS

PRESENTED JUNE, 1907



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April 30, 1907.

This is to certify that the following thesis prepared under the immediate direction of Professor F. O. Dufour, Assistant Professor of Structural Engineering, by

JOHN ALBERT CALLAN

entitled      DETAILS OF HIGHWAY BRIDGES

is accepted by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

----- *Ira O. Baker* -----

Head of Department of Civil Engineering



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## THESIS.

### INTRODUCTION.

A chain is no stronger than its weakest part, and in this respect may a bridge be said to resemble a chain. Not only one, but all parts of a bridge must be well designed. These parts must not only be sufficiently strong but they must also be economical. It is to determine the strongest and most economical details, that this thesis has been written.

While much has been written concerning the determination of the stresses in the various parts of a bridge, but little has been said in the way of comparing the most efficient designs to withstand stresses. It is therefore the purpose of the writer to show the relative merits of the most common designs of the details of highway bridges. The more common types will usually be first introduced with a discussion of their relative merits, after which the less commonly used details with their merits and defects will be discussed. Shop drawings of ninety four highway bridges built by thirteen representative companies of the United States, were investigated. Most of these bridges were Pratt pin-connected trusses and of spans varying from 70 to 273 feet in length. Sketches are shown wherever necessary to assist in making the discussions more clear.



## ART. 2. END POSTS AND UPPER CHORDS.

The end posts and upper chords of a highway bridge are usually made up of two channels laced, or of two channels and a cover plate, the channels being laced on the under side. In cases where lacing alone is employed, stress in the members due to eccentricity of the pins is reduced to zero when the pins are on the center line of channels, and thus extra material is saved.

Of the ninety-four highway bridges investigated, forty-six had cover plates. Cover plates are always employed for large spans on account of the added rigidity which they furnish the member.

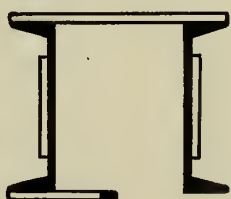
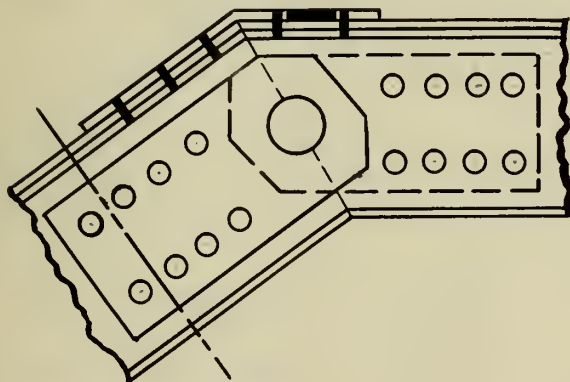


Fig. 1.

One of the greatest disadvantages of the cover plate is that it causes eccentric stress in the chord members, due to eccentric bearing on the pin which is in many cases placed on the center line of the channel. This difficulty may be obviated by setting the pin



at the center of gravity of the section. This should always be done when the depth of the channels will permit.

The use of the cover plate is to be encouraged because it not only stiffens the members, but also adds both to its protection against the elements and to its appearance. Of the bridges investigated, 52.0 per cent of the end posts and upper chords were made up of channels, lacing, and a cover plate.

The section of the end post and the upper chord is usually made the same. Of the bridges investigated, 63.6 per cent had the sections of the end posts and upper chords the same. This method causes uneconomical efficiencies in the chord members since, while the efficiency of the end post is a minimum, for the panels of the upper chord nearest the end post it will be a maximum. The extra cost of material to obtain this latter maximum efficiency is more than offset by the simplifying of such details as rivet spacing, lacing, splicing, and the lessened cost of the templet and shop work.

The channels for the chord members should be spaced so as to cause the moments of inertia about the horizontal and vertical axes to be equal, in order that the sections be economical. This however rarely obtains since the cover plate would usually become too narrow for use in the packing of eye bars and vertical posts.

Lacing on the under side may be single or double. If sin-



gle, most specifications require that the angle the lacing makes with the longitudinal axis of the member be not less than sixty degrees, while for double lacing the angle required is forty-five degrees. Nevertheless, in all the cases investigated, single lacing inclined at an angle of forty-five degrees was used wherever a cover plate was present. This may be still satisfactory, notwithstanding specifications, since the cover plate has a greater effect towards stiffening the member than that produced by double lacing. Wherever double lacing is employed in place of a cover plate lacing bars should be riveted at their intersection as this aids in stiffening the member, since the size of lacing bars cannot accurately be determined on account of the fact that it is impossible to accurately determine the shear. The size of the lacing bars are usually obtained from specifications which are the result of many years of experience.

According to Cooper's specifications, lacing bars should have a thickness of not less than one-fortieth in single lacing, and not less than one-sixtieth in double lacing, of the distances between the rivets connecting them to the main members.

In order to prevent buckling, the ends of all laced members should be stiffened by means of batten plates which should not be less than  $3/8$ -inches thick. Cooper's specifi-



cations also state that all batten plates should never be less in length than one and one-half times their width, but in nearly all cases investigated the batten plates were less in length than their width. This is satisfactory for all cases where a cover plate is used since the cover plate adds sufficient stiffness to the members. It would not be satisfactory however for those members in which no cover plate is used.

Batten plates should be placed as near the end of the member as possible in order to protect the pin connections from the weather.

All pins for the upper chord should be thoroughly packed to keep chord members and diagonals in their proper positions

The ends of all members of the end post and upper chord should be accurately milled in order that the abutting joints bear evenly against each other at the ~~the~~ splices. Cooper states in his 1901 specifications, that all splices should be designed to transmit all of the stress, but this would be unnecessary if the joints were accurately milled for then, theoretically and practically, all of the stress could be transmitted through the splice without the need of a splice-plate. Splice-plates should never be less than  $3/8$ -inches in thickness. Chord members should be reinforced by pin plates attached to each channel at the pin connection in order that the allowable bearing stress on the pin be not exceeded.



Pin plates should be long enough to allow their portion of the stress to be transferred to them by the rivets. For the end post and the upper chord connection, the pin plate should extend 6 inches beyond the batten plate in order to stiffen the members.

### ART. 3. INTERMEDIATE POSTS.

Intermediate posts are usually made up of two channels and lacing and are of two general types namely, those whose channel webs are placed parallel to the direction of the roadway, and those whose channel webs are perpendicular to the roadway. Of the ninety-four bridge designs examined, fifty-seven had the webs of the channels parallel to the direction of the roadway, while the remaining thirty-seven had the webs of the channels perpendicular to the roadway. In other words 60.6 per cent of the bridges investigated had channels that were parallel to the roadway. Chief among these was a 273-foot bridge built over the Iroquois River by the Massillon Bridge and Iron Co.

The placing of channels with their webs parallel to the roadway is superior to the placing of the webs perpendicular to the roadway for the following reasons:

1st . It admits of having the chord pin pass through the webs of the channels, and so does away with the heavy pin-



plates that are necessary when channels have their webs perpendicular to the roadway.

2nd. It admits of a better riveted connection to the upper chord, since the connecting plate in this case may have almost any number of rivets in them, since the connection is made through the webs of the channels for both members, while for connections with channels whose webs are perpendicular to the roadway either only the flange of the channel can be used for riveting, thereby losing the stiffness developed in the former case, or a pin-plate which is not desirable is required for the connection.

3rd. It admits of a better connection to the floor-beam hanger, since angles may be used to connect the web of the floor-beam to the web of the channel, thereby stiffening the connection; whereas for channels with their webs placed in the other direction, only the flange can be used. This last named advantage is offset by the fact that the stress in the floor beam is not equally distributed to the posts. Also, in the majority of the cases where this form is used, the flanges have to be chipped off in order to pack the joints well, thereby making the form expensive. However, the channel flanges could be turned in to advantage.

4th. It does not require the web of the channel to be cut in order to admit the counter tie being placed about the middle of the pin, which is the case for all bridges of an even number of panels and having the channel webs of the intermediate posts perpendicular to the roadway.

The placing of the channels with their webs perpendicu-



lar to the roadway is advantageous because:-

- 1st. The stress in the floor beam is distributed equally to both channels.
- 2nd. It affords a simple connection to the hub guard.
- 3rd. It admits of well packed joints.
- 4th. It admits of a good floor beam connection, which is shown in Fig. 72 Page 74. Therefore from the preceeding discussion it would seem that the method of placing the channels with their webs parallel to the roadway is the best for trusses varying from the smallest to medium length spans of about 130 feet, while the placing of channels with their webs perpendicular to the roadway is best for large spans.
- 5th. It admits of a larger moment of inertia being developed about the axis perpendicular to the roadway, by the placing of the channels farther apart. This is good since the purpose of the truss is to oppose the flexural stress developed by the dead and live loads in the bridge. Therefore any step taken in giving this flexural stress a larger distance to act through, will be a step taken toward the absolute stiffening of the member.

The lacing of intermediate posts is identical with the lacing on the under side of end post and upper chord since the specifications are the same for each. For double lacing, the angle which the lacing makes with the longitudinal axis of the post should be forty-five degrees. Figures 2, 3, and 4 show some of the most common connections of intermediate posts to upper chords.



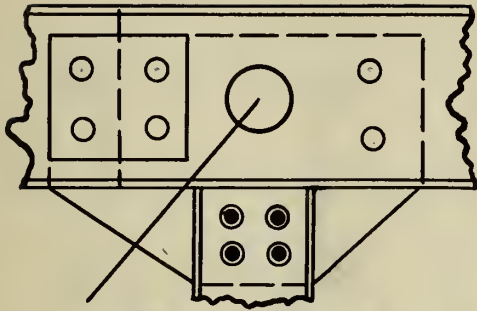


Fig. 2.

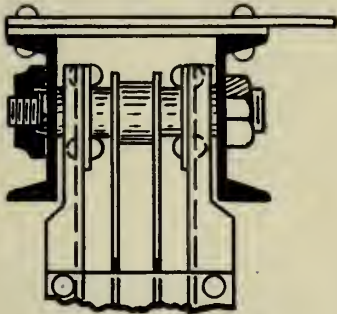


Fig. 3.

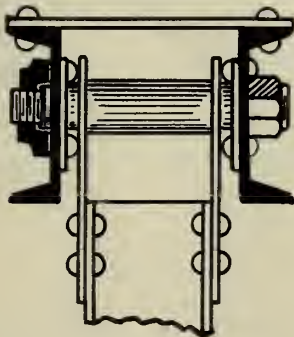


Fig. 4.

Fig. 2 shows the most common type of connection of intermediate post to upper chord where channels are parallel to roadway. Fifty-seven cases of this type, out of the ninety-four investigated being found, while only two cases of Fig. 3 were found. The connection shown in Fig. 3 is however superior to that of Fig. 2 since it would require no field riveting and would facilitate erection. It would however be more costly on account of the cutting of the channel flange. The connection of Fig. 2 is faulty since it requires field riveting, and the plate is liable to become distorted and bent during transportation. Its use generally lies in the fact that it assists in the splicing of the upper chord.

The connection shown in Fig. 3 is always used where the channels have their webs perpendicular to roadway, and is as efficient a method as can be devised in this

case, although it throws a great amount of stress to the flanges



of the channels.

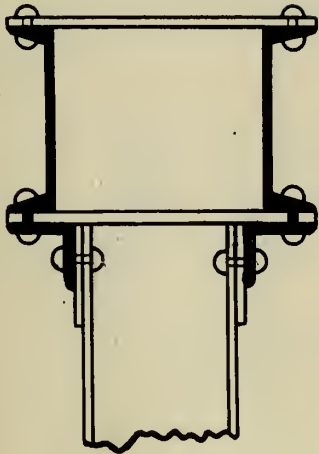


Fig. 5.

Fig. 5 shows another method of connection of intermediate posts to upper chord for channels where webs are perpendicular to roadway and was found in only one case. The bridge was 105 ft. span, and was built by the Lafayette Bridge Co., in Champaign Co., Ill.

The connection is not an improvement over the preceding one. The riveting must be done in the field and there is no saving in the amount of material required. Then too, unless riveting is done very accurately, some eccentric stress will be set up in the chord member. It is not as efficient a connection as that shown in the preceding case.

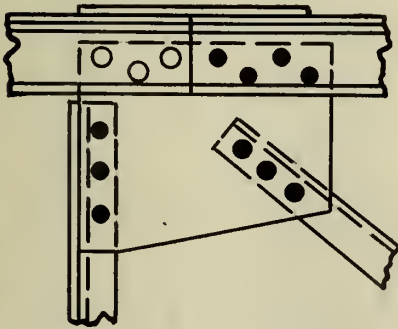


Fig. 6.

Fig. 6 shows a form of connection of intermediate post and upper chord for a 78-ft. span pony riveted-truss. The intermediate post consists of two angles fastened together by means of a cover plate. Angles could not be used for large spans. The connection is poor on account of the eccentric stresses that may be set up in the chord

member. Then too, the large plate is liable to become distorted during transportation.



ART. 4. THE HIP VERTICAL.

There are four general forms of hip verticals in use, namely:

- 1st. Half-post construction where channel webs are parallel to roadway and connected to upper chord by means of loop or eye-bars.
- 2nd. Half-post construction where channel webs are perpendicular to roadway and connected to upper chord by loop or eye-bars.
- 3rd. Half-post construction where channel webs are parallel to roadway and loop or eye-bars extend between the pins at  $U_1$  and  $L_1$ .

For very small spans, the hip vertical may be dispensed with, but for spans of fifty feet or more, it should always be used.

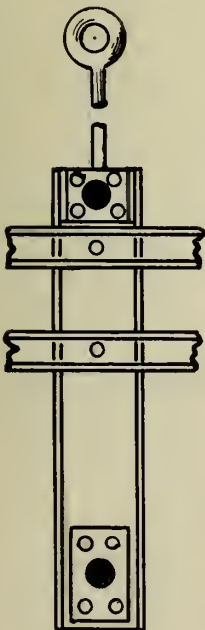


Fig. 7.

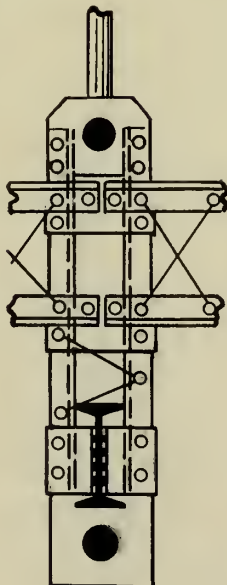


Fig. 8.

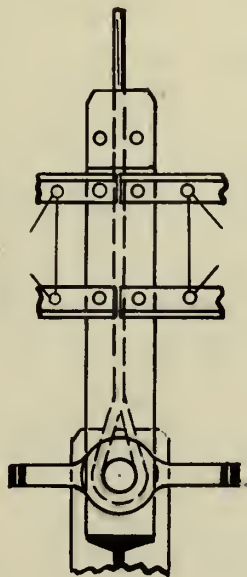


Fig. 9.

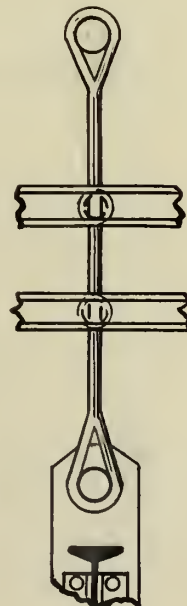


Fig. 10.



Of the ninety-four bridges investigated, twenty-five were of Class One, twenty-two of Class Two, thirty-nine of Class Three, three with no hip vertical at all, two were built up intermediate posts whose channel webs were parallel to the roadway and three consisted of only two bars with clamps by which the hub guards were attached. The above types are shown in Figs. 7,8,9, and 10.

Although Class One was not employed in the greatest number of cases, yet it is nevertheless a very efficient mode of connection for the following reasons:

- 1st. Chord pin may be passed through the web of channels thereby involving no use of pin-plates as is necessary in Class Two.
- 2nd. It equalizes to a great extent the stresses in the bars above to be transferred from the floor beam for reasons which are explained on Page No. 59 under floor beam connections.
- 3rd. It furnishes a stiffer connection than Class Three.
- 4th. It does not require a special construction for the support of the hub guards and sidewalks.

Although Class Three was employed the greatest number of times, it is not the most efficient mode of connection for the following reasons:

- 1st. It requires a pinplate in nearly all cases.
- 2nd. It requires packing and fillers for the loop bars which tend to get out of line.
- 3rd. It requires a construction for the support of the hub guards and sidewalks in most cases.



4th. It causes eccentric stress in rod as a result of attaching the floor beam.

Class Two is used almost as much as Class One, and is a very efficient method of connection for the last three reasons stated under Class One.

Only very small highway bridges of spans of fifty feet or less can be used without hip verticals, since the hip vertical has to support both a dead and a live panel load when loaded to a maximum. The building of the hip vertical of the same section as the intermediate posts is uneconomical as it involves a very high efficiency. It should not be allowed on any highway bridge unless of sufficiently large span to demand a more economical hip vertical than here described, while for very small spans the hip vertical may be left out.

Fig. 10 shows a form that is objectionable for any but very small spans. For small spans it is efficient as the stress in the hip vertical is very small indeed and the panels are usually of sufficient shortness to allow the hub guards to be efficiently supported between the end posts and the first intermediate post.

Fig. 11 shows a form of hip vertical employed on a pony riveted truss of 76-foot span. The length of the hip vertical was 9 feet. It is efficient for small spans, but would not do for large spans since it is very weak about an axis perpendicular to the roadway. For larger spans, where a top lateral system could be used, it would be better to place angles



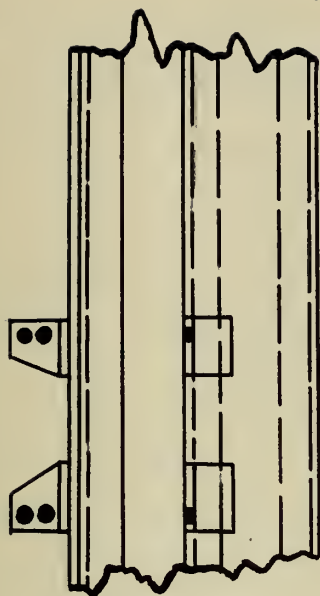


Fig. 11.

in a direction parallel to the roadway in order that the hip vertical can stand the flexural stress successfully. For small spans however it is very efficient since it, by its stiffness in a direction perpendicular to the roadway, makes up for the lack of a top lateral system.

#### ART. 5 . MAIN AND CCUNTER TIES.

Main and counter ties are usually made up of loop or eye-bars. The difference between a loop and eye-bar is that in the former the loop is welded, while the loop or eye in the latter is formed by upsetting. Owing to imperfection in workmanship, the eye-bar is the best.

The most economical proportion between the width and depth



of bars for the smallest allowable pin in an iron bridge can be determined from the following discussion taken from Waddell's (Designing of Highway Bridges).

"For one pair of bars acting on the pin, and the tension considered a fixed quantity, the stress in one bar equals  $WDT$  and moment  $= W^2DT$  which should be equal to the resisting moment which is  $\frac{SI}{c}$ ,  $W$  being the width of the bar, and  $T$  being the unit tensile stress. In this case,  $S$ , the allowable unit stress due to moment  $= 1.5T$  in pin,  $I = 1/4 r^4$  and  $c = r = d/2$  where  $d$  = diameter of pin and  $r$  = radius of same.

Substituting above values in equation  $M = \frac{SI}{c}$ , we get  $M = 3/64 Td^3$ . Equating the two values of the moments,

$$W^2DT = 3/64 Td^3 \text{ or}$$

$$W^2 = 3 d^3/64 D.$$

Since to make  $d$  a minimum,  $W$  must be a minimum and the smallest possible section is to be employed, this being obtained by computation from the allowable unit stress and a maximum stress required to be met, and, since  $WD =$  a constant,  $W$  will be smallest when  $D$  is largest.

Let  $C$  = allowable working compressive stress. The compression on the pin and eye should equal the tension in the bar.

$$\text{Compression on pin and eye} = WdC.$$

Then  $WDT = WdC$ . Since  $C = 12000$  lbs. and  $T = 10000$  lbs., there results the equation  $d = 0.833 D$  or approximately  $0.8 D$ . Transposing  $D = 1.25d$  which = maximum relation of  $D$  to  $d$ .



Substituting in equation at preceding page,

$W^2 = 3 / 64.64 / 125 D^2 = 0.754 D^2$  and  $W = 0.274 D$  or in other words, the depth of the bar = about 4 times the width.

Following out the same process for 2 bars,  $W = 0.194 D$  which means a ratio of depth to width of 5 to 1.

For three pairs,  $W = 0.159 D$ ,  $D =$  about 6  $W$ .

For four pairs,  $W = 0.137 D$  and  $D =$  about 7  $W$ ."

Chas. Mc Donald, C. E., in his paper read before American Society of C.E. in 1874, has shown for flat eye bars,  $D = 1.721 t \sqrt[3]{n}$  where  $t$  equals thickness of bar,  $n$  equals width of bar and  $D$  equals diameter of pin but no allowance was made for the number of bars acting on pin. For round bars  $D = \sqrt[3]{4d^3 - 1/2 d^2}$  where  $d$  equals diameter of bar.

To sum up; for three pairs of bars acting on a pin at its ends, such as will occur at  $L_1$ , the ratio of the width to the depth should be as 1:6, while for four pairs of bars acting on the pin at  $L_2$  or  $L_3$ , the ratio of width to depth should be as 1:7. This however is never followed out in practice since it seems to be the desire of all designers to spend as little time as possible on the design of the main and counter ties, their only aim being to obtain an area of cross-section as near equal to the required area as possible, no attention being paid to the ratio of the width to the depth; in fact square bars are often used. However in most cases a ratio of 1:4 or 5 is employed.

Two bars should always be used for counter purposes and these should be placed as near the other members as is



possible, in order to reduce the bending moment to a minimum. This however is seldom done except in odd-paneled trusses.

All counters should be equipped with either a turnbuckle or sleeve nut to be efficient in their purpose, and not throw any undue stress on the main ties and thereby cause same to buckle. Counters should, in all cases, be packed thoroughly, whether channel webs be perpendicular or parallel to roadway, in order to prevent any eccentric stresses. In a large number of cases, packing is dispensed with, especially when channel webs are perpendicular to roadway, as designers generally leave the channel web to hold the counter tie in place. This is not good practice for reason stated above. Fig. 12 illustrates this.

The packing of ties will be considered in the succeeding article.

Fig. 12 Shows a peculiar method of attaching main and counter ties to intermediate posts, thus making an ordinary Pratt truss with broken upper chord, into a modified camel-back Baltimore truss.

Two samples of this type were found, one on a 273-ft. span bridge built by the Massillon Bridge Co. over the Iroquois River, the other on a

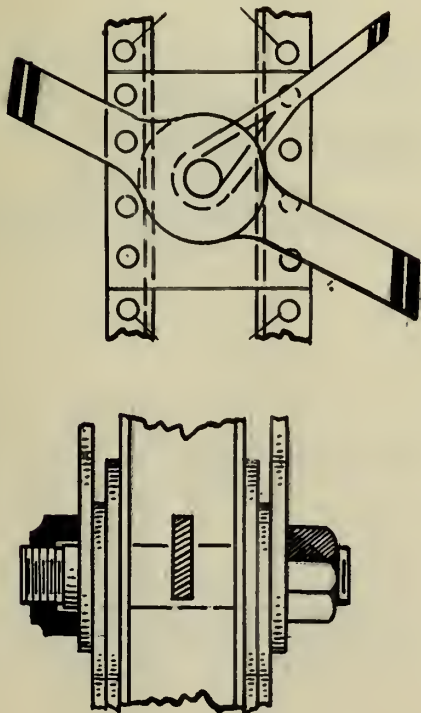


Fig. 12.



109-ft. bridge, built by the Lafayette Bridge Co., over the Sangamon River, in Monticello Twp., Ill.

Wadell, in his "Designing of Ordinary Iron Highway Bridges", states that the size of pins in double intersection bridges where the diagonals are halved and are coupled on pins passing through the middle of the posts, should be found from the moment  $M = SW/2$  where  $S$  is the stress on the diagonals and  $W$  the width of one of the main diagonals.

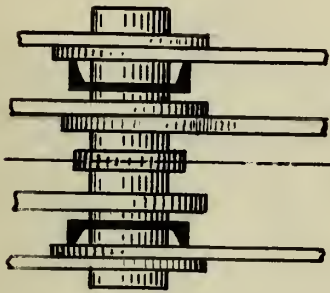


Fig. 13.

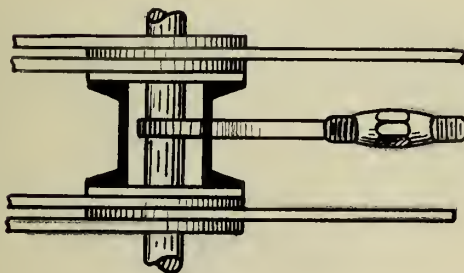


Fig. 14.

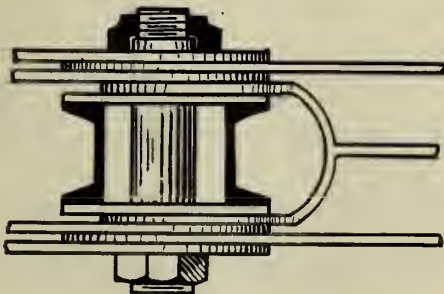


Fig. 15.

Figs. 13, 14 and 15 show the most common arrangements of main and counter ties. Of the bridges investigated, fifty had a single counter placed on the center of the pin, three had single forked counters while five had the single counter placed in an unsymmetrical position.

Fig. 14 in which the counter is placed at the middle of the pin represents by far the most common method employed. It is economical where the channel webs of the posts are parallel to the roadway, but for the type shown in Fig. 13 it is not, on account of the cost required to cut the channel webs. The form is on the whole not good, since it



allows a very large bending moment to be developed in the pin.

Fig. 13 shows a form which was found in but very few cases, and while it does not cause the large bending moment to be developed in the pin, it causes an unsymmetrical loading on the pin and thereby produces eccentric stress in the post. It therefore cannot be considered an improvement over the form shown in Fig. 13.

Fig. 15 shows the most efficient form of the three,

although it is the least used. it causes no eccentric stress in the post, neither does it cause a large bending moment in the pin. It is however more costly than the preceding two forms discussed, on account of its peculiar shape and this fact probably accounts for the lack of its use. Two counters should always be used if possible, but if only one is

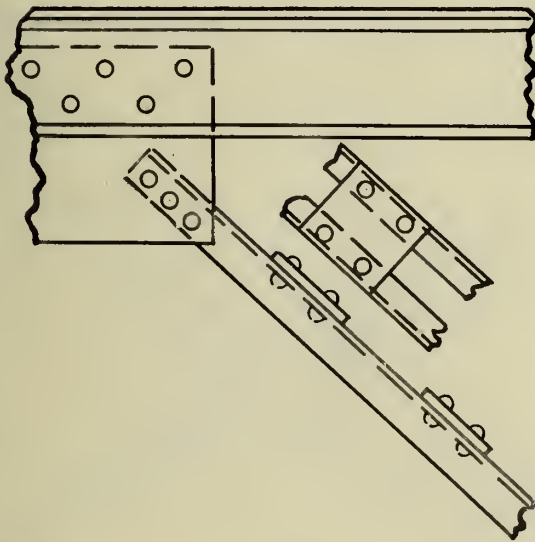


Fig. 16.

used this one should be of the form shown in Fig. 15.

Fig. 16 shows a form of main and counter tie employed in a 76-ft. span pony riveted-truss of five panels. The tie consists of two angles fastened together at intervals by means of batten plates. The form is very efficient indeed for the place that it serves since it is economical, rigid, and strong. For long spans, channels might be used in place of angles.



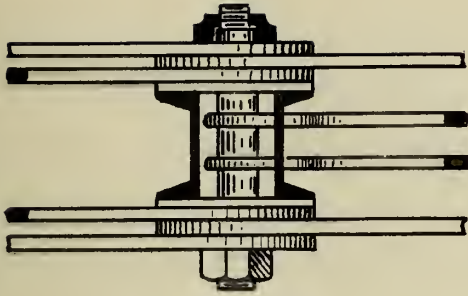


Fig. 17.

lengthened and the two counter ties placed on the outside of the cover plate of the two channels in order to reduce the bending moment on the pin. Also, it is too expensive since it requires the channel web to be cut in two places.

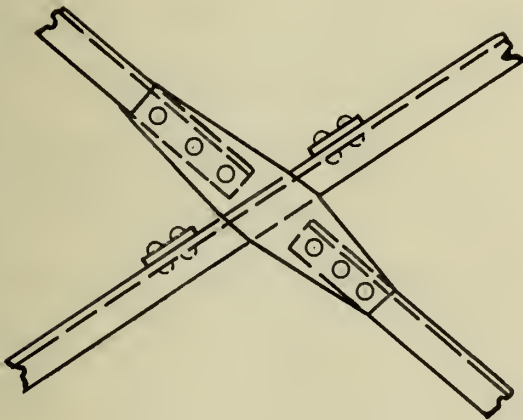


Fig. 18

be avoided if the plate is shipped loose.

Fig. 17 shows a method of main and counter tie connection employed for bridges of an odd number of panels, with channel webs of posts perpendicular to roadway. It is not a good connection, since the pin should be

Fig. 18 shows the method of crossing of main and counter ties in the same bridge as form of main tie shown in Fig. 17 belongs. The connection is efficient but is not desirable since the plate is liable to become bent and distorted during transportation. This fault can

#### ART. 6. LOWER CHORDS.

Each and every lower chord member of a pin connected highway bridge is usually made up of either two eye or two loop bars, eye bars being preferred. The preceding discussion under Art. 4, shows how the proper proportion between width and depth of bars is determined for the smallest allowable pin.



In general, the ratio of the width to the depth of bars of the lower chord is the same as that in the main ties.

Wherever a number of tension members are acting about a pin, they should be alternately placed on each side of the pin to avoid a large bending moment. They should also be packed as close as is possible for the same reason.

Practice seems to vary as to the placing of the chord members on the pin, but the best type was seen in the majority of the bridges investigated, where the chord member nearest the middle of the truss was placed on the outside. The other chord member was then placed next, the diagonal nearest the middle of the truss in odd paneled trusses was then next, the diagonal farthest away from the middle of the span being last.

For bridges having the posts with the webs of the channels parallel to the roadway, the post is usually placed immediately within the chord members, while for those having the webs of the channels perpendicular to the roadway, the post is placed with the pin between and parallel to the channel webs. Pins on the lower chord should always be so packed as to allow the various members acting on them to reach their maximum efficiency.

The lower chord for rivet connected trusses is usually made up of angles held in position by batten plates or lattice bars which are placed at intervals along the length. The lower chord is at once a rigid member. The form is both economical and efficient, and is therefore recommended on account of its simplicity in construction and rigidity. The



connections are also simple and economical as can be seen in Fig. 19.

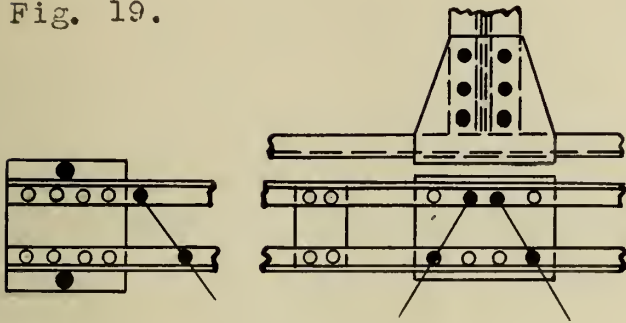


Fig. 19.

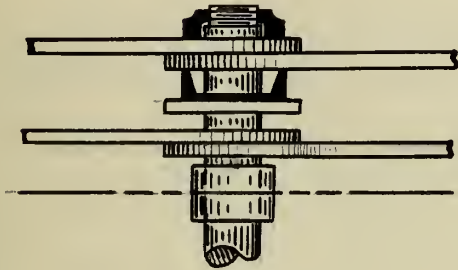


Fig. 20.

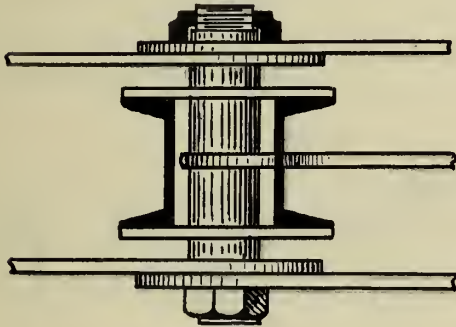


Fig. 21.

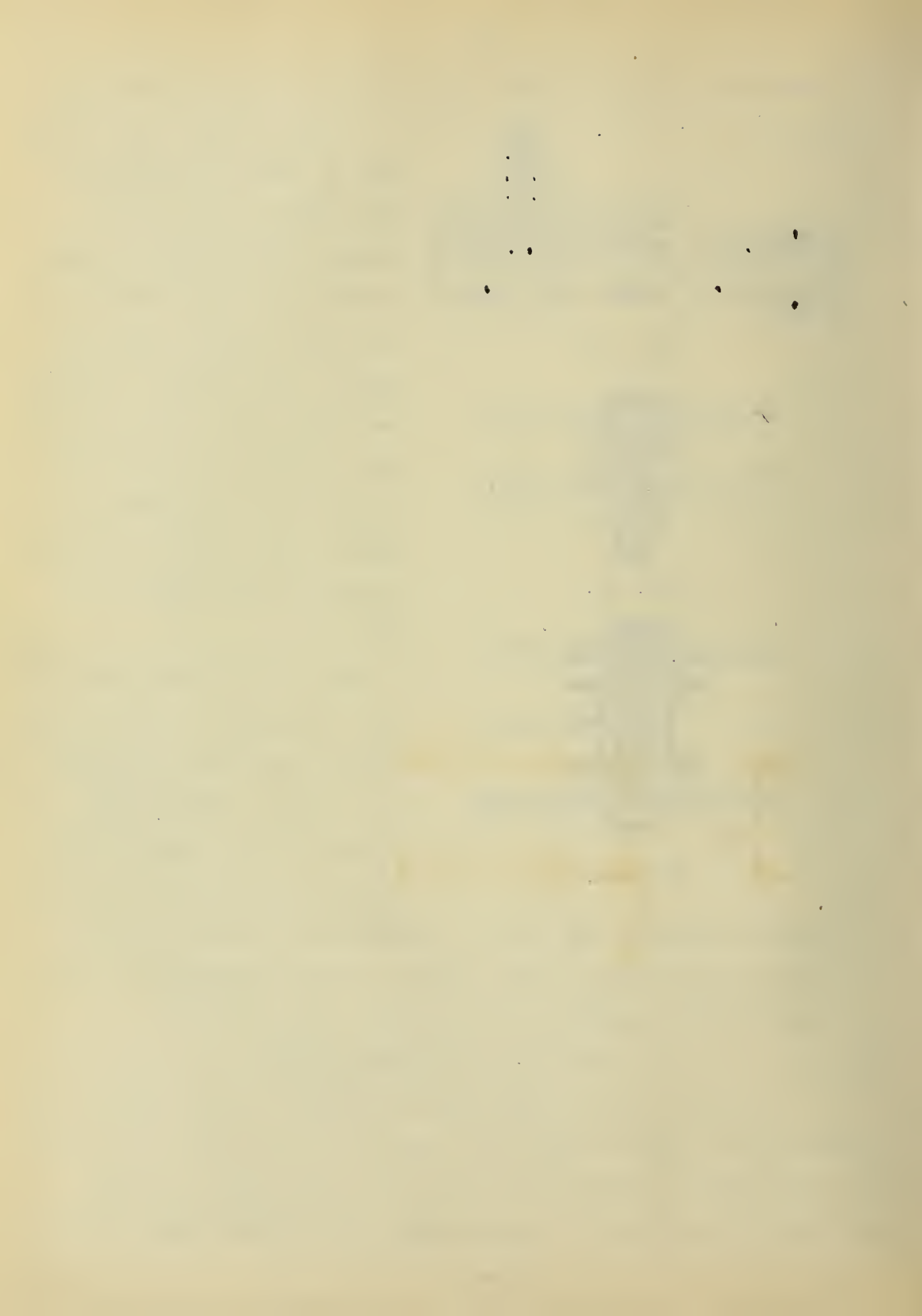
Fig. 19 shows the arrangement of the various members upon the pin, and also the packing employed for an odd-paneled truss with posts having their channel webs parallel to the roadway. It represents the most common practice for that type of truss. For trusses with even number of panels, the counter is usually placed in the middle of the pin.

Fig. 20 shows the most common arrangement of the various members upon the pin, for an even paneled truss having posts with the channel webs perpendicular to roadway. For

trusses having an odd number of panels, two counters are used, and they are placed either next to the chord members or on the inside of the channel as shown in Fig. 16, Page 20.

#### ART. 7. TOP LATERAL STRUTS.

There are a very large number of types for top lateral struts, nearly every bridge possessing a type of its own. Some of the types are very unstable however, and possess but very little stiffness but they are however, as a general rule, less costly than some of



of the commoner types and are only used on small spans.

For long span highway bridges, the top lateral struts should be made very rigid or the rigidity of the former is lost in the latter. The top lateral strut should be designed to resist both tension and compression. The radius of gyration about an axis parallel to roadway should be very large in order to insure stiffness.

Batten plates should be used in all cases where possible, and should be placed near the end of the member.

For long spans, lacing should be double, but for short spans, single will suffice. Single lacing should, and usually does, make an angle of sixty degrees with the longitudinal axis of the strut while double lacing always makes an angle of forty-five degrees with same axis.

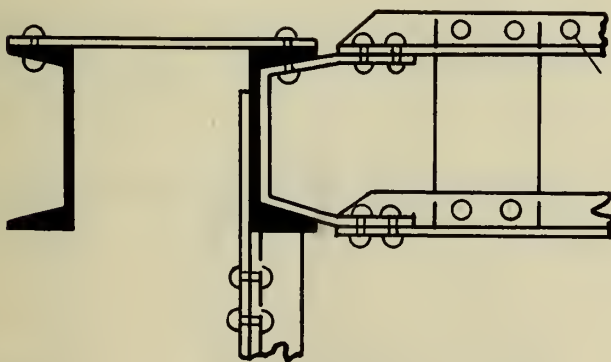


Fig. 22.

Fig. 22 shows a very common type of strut. Of the bridges investigated, twenty-four had struts of this type. The strut is sufficient and, since it has batten plates at each end, is also stiff.

Then too, it develops no bending moment on the pin and its form is especially fitted to take compression. It is however uneconomical for large spans on account of the thickness of the bent plate



which must be made quite large to withstand the large stresses coming upon it. It is not recommended for spans of over 100 feet in length.

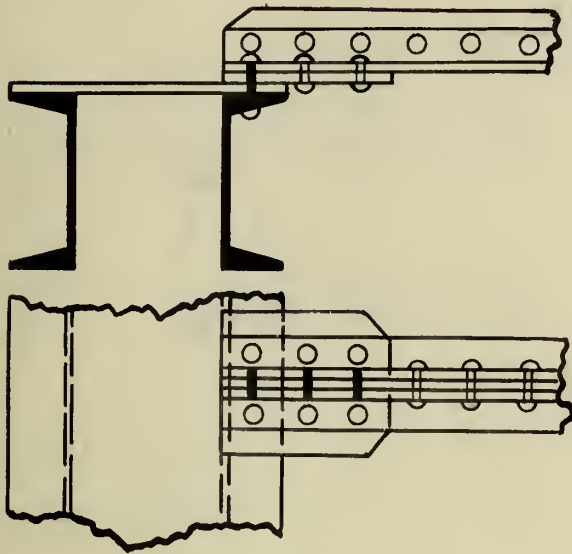


Fig. 23.

angles were made to extend the full distance across the upper chord member.

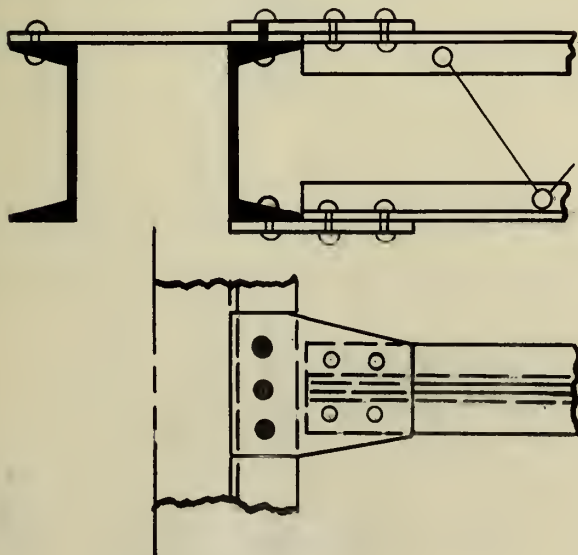
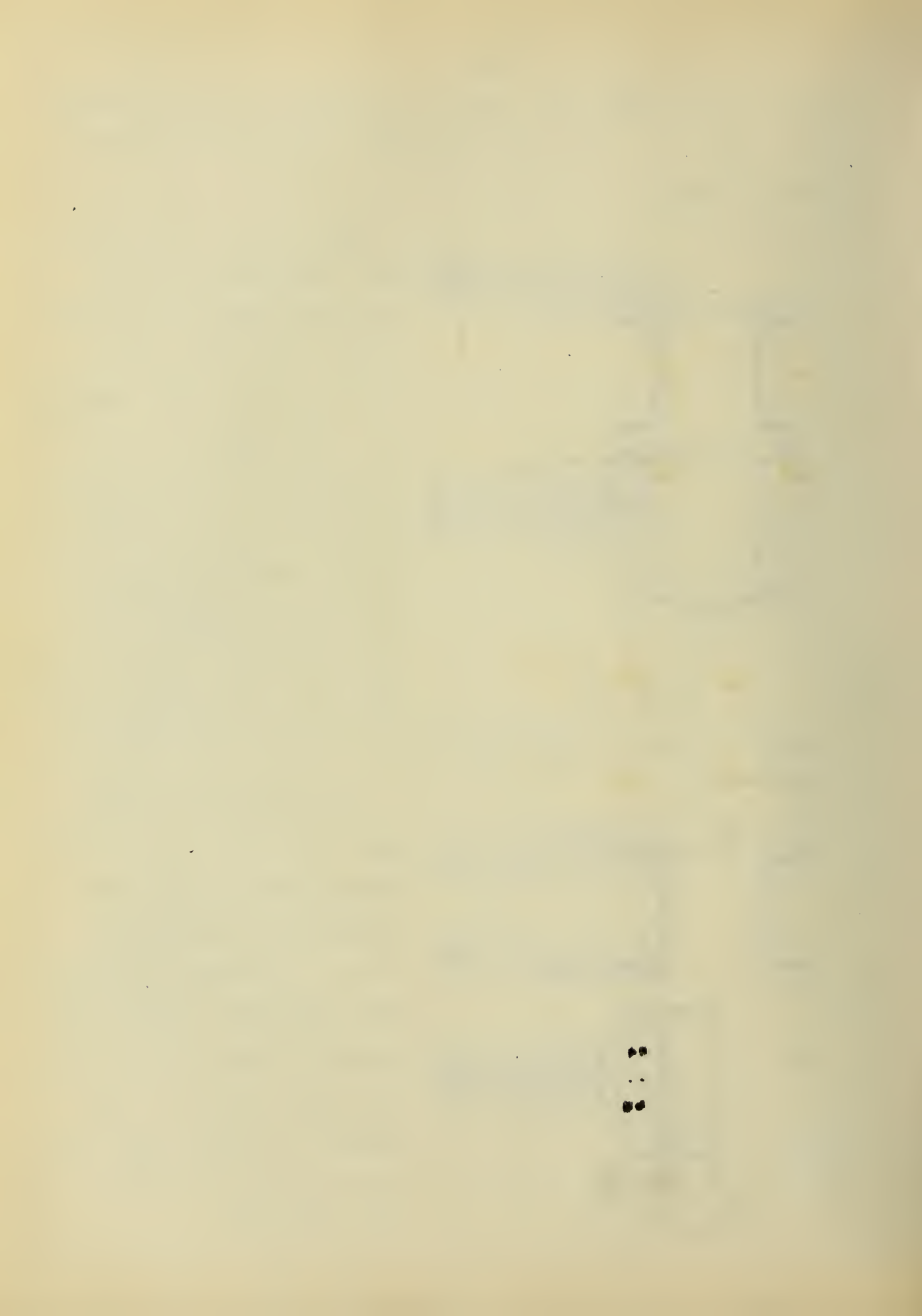


Fig. 24.

Fig. 23 also represents a very common type of strut employed, twenty-four types of this kind being found. It is, however, only employed for small spans for which it is both a very efficient and economical type, since but little material is required and the stresses developed are not large. It could be used for longer spans if the

Fig. 24 represents the next most common type of top lateral strut found, nine cases of this type being seen. It is both a simple and economical type for the smaller spans, but for longer spans the upper angle should run clear across the cover plate as is seen in Fig. 28, p.26.



It is to be recommended for spans up to 100 feet in length. The type could be improved upon if the lower angles were turned down as the upper ones are, as less surface of the angles would then be exposed to the elements.

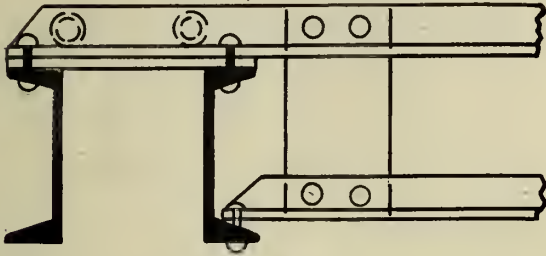


Fig. 25.

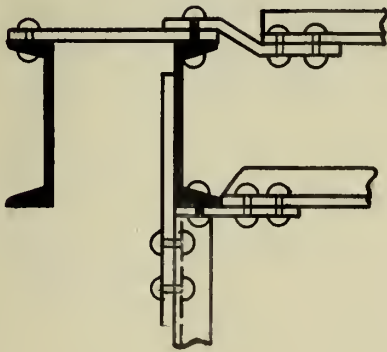


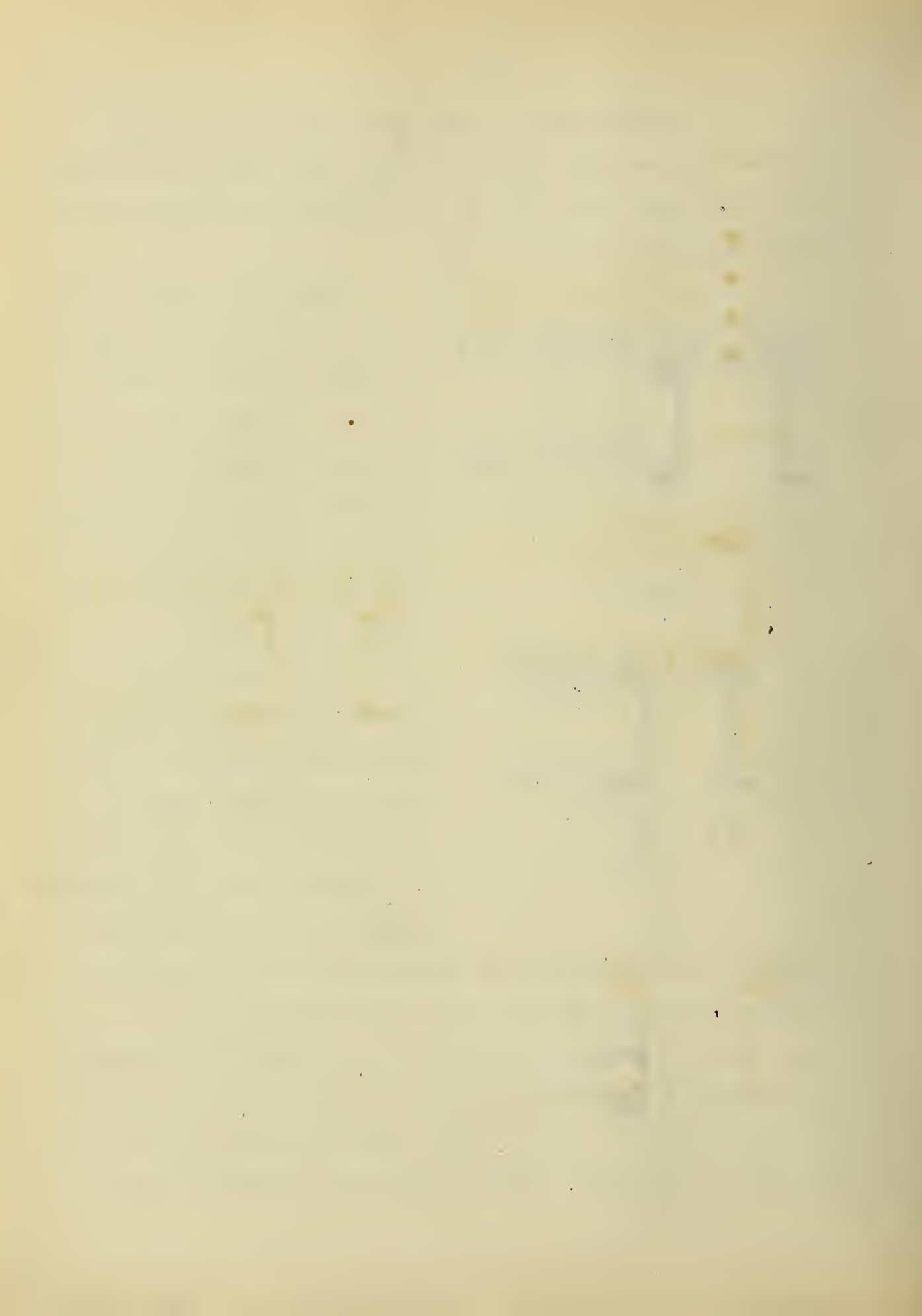
Fig. 26.

A very rare type of connection of top lateral strut to chord members is shown in Fig. 25. It was found in only two cases. The fastening of the lower angles to the flanges of the channel is not efficient and the form is therefore not to be recommended.

Fig. 26 also shows a very inefficient type, this type being found in but one case. While no bending moment is developed on the pin, the cost of the bent plate, its weakness due to the position of

its load and its liability to become distorted during transportation, prevent the type from being recommended.

Fig. 27 shows an efficient mode of connection employed in a bridge of large span. The hole shown at a, is to admit the rod used in the sway bracing. The top plate should extend clear across the chord member. The type is expensive and is



to be avoided wherever possible.

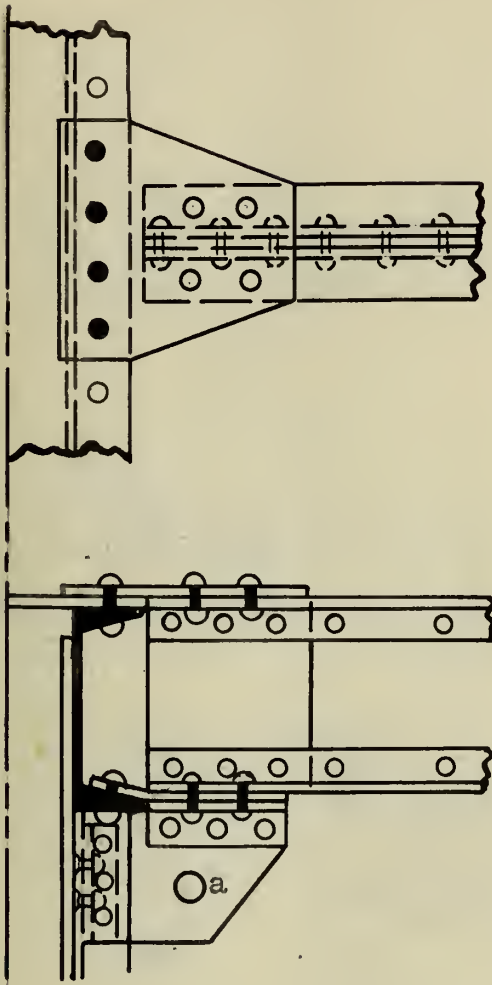


Fig. 27.

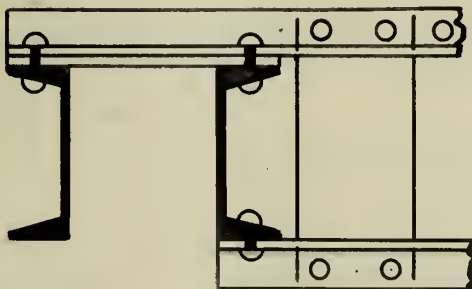


Fig. 29.

Fig. 28 shows a very efficient mode of connection. It was found employed in eight cases. It may be used for spans of any ordinary length. It could be improved upon as mentioned in discussion of Fig. 24, which is shown on p. 24.

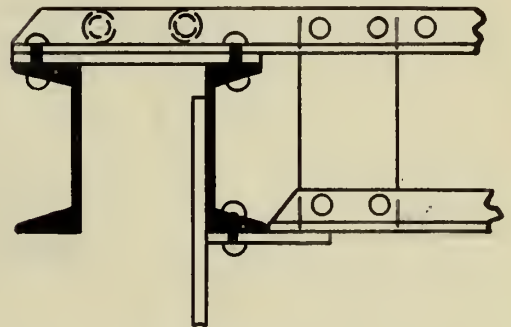
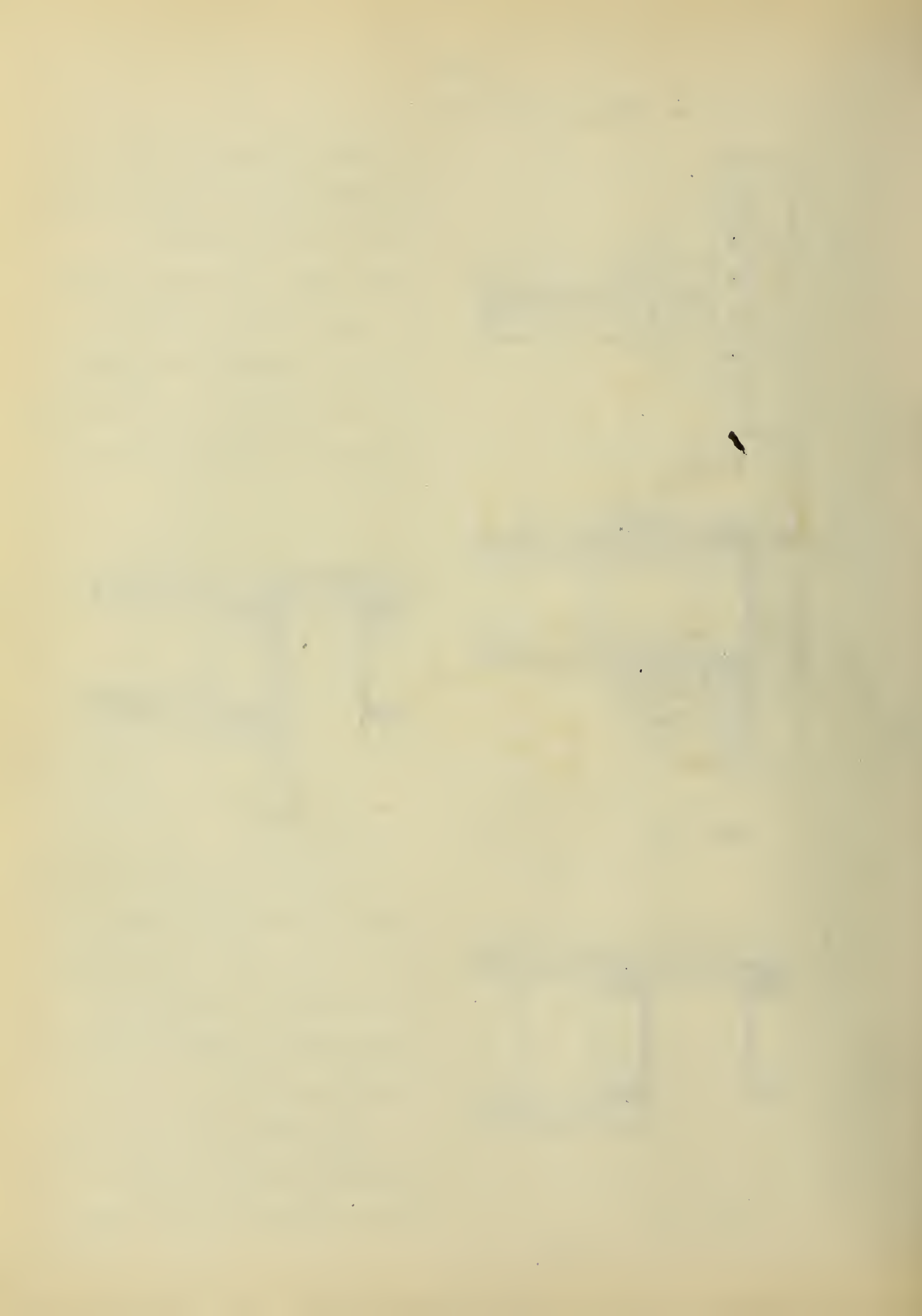
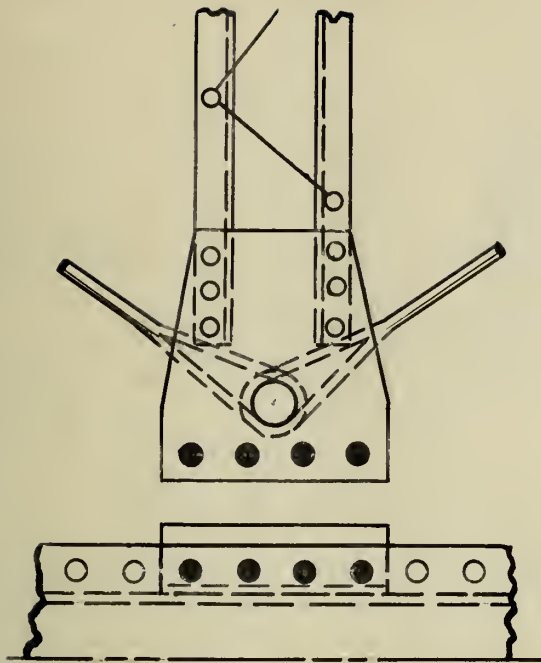


Fig. 28.

Fig. 29 shows a form of connection which was found in only three cases. It is also efficient for reasons mentioned under discussion of Fig. 26, p. 25.

Fig. 30 shows a form of connection which was only found employed in one case.





This was in the Oakland bridge, made by the Chicago Bridge and Iron Co., the span being 180 feet. The form is efficient for its purpose, but is quite expensive. It was used in order to furnish good connection for the sway bracing which was employed in that bridge.

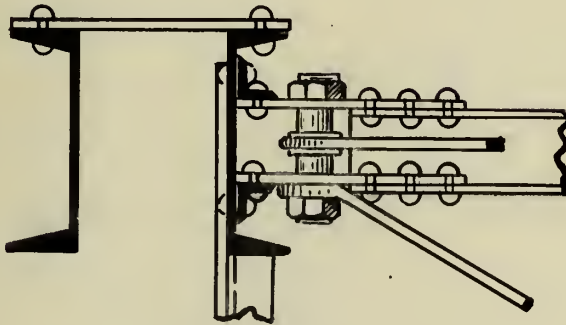


Fig. 30.

#### ART. 8 . TOP LATERAL DIAGONALS AND CONNECTIONS.

A first-class top lateral diagonal should possess the following features:

- 1st. It should be economical in section.
- 2nd. It should offer an efficient and economical method of connection to the chord member.
- 3rd. It should possess sufficient stiffness.

It should be economical in that it should furnish no



more material than is actually required and should be easy to erect.

It should offer an efficient and economical method of connection to the chord members to prevent the loss of any of the rigidity of the diagonal without incurring extra expense.

It should possess sufficient stiffness in order to serve its purpose fully and prevent any eccentric stresses in the chord members.

Top laterals are of two forms, namely:- rods and angles. The majority of lateral diagonals are composed of rods although in many respects, angle bars are to be preferred.

The general use of rods lies in the facility with which they may be put in place. Cooper's 1901 specifications require that they be able to resist both tension and compression, but his specifications are not lived up to in this particular for rods can take but very little compression.

The use of rods, however, is not so faulty if proper precautions are taken. These precautions are, that a strong and economical connection be used and an efficient method of tightening the diagonal be offered. But very often an inefficient uneconomical method of attachment is offered and rods are allowed to sag, thus causing eccentric stresses in chord members. These stresses may become considerable, and so reduce the efficiency of the diagonal members. An angle bar on the other hand, is deeper and, therefore, much stiffer, and does not tend to sag. Also it will take much more compression than a rod. Also, the connection with the chord member is much simpler



and therefore more economical since an angle bar only demands sufficient space for attachment by the required number of rivets.

It is a curious fact that in nearly all cases angle diagonals are only used for very small span trusses; a case where a first class diagonal system is least needed. The advocates of the use of rod diagonals claim that if a sufficiently strong and stiff top lateral strut be used, the use of an elaborate system of diagonals is not necessary.

Cooper's 1901 specifications state that the top lateral diagonals of a through truss should be designed for a uniform wind load of one hundred and fifty (150) lbs. per linear foot, while for a deck truss the top lateral diagonals should be designed, in addition to the above, for a moving wind load of four hundred and fifty (450) lb. per linear foot, the allowable unit stress for both static and moving load being the same, or eighteen thousand (18,000) lb. per square inch.

The following figures show some of the most common forms of connections employed, rods being used in every case. Of the bridges investigated, twenty-three had connections similar to Fig. 31, sixty-five to Fig. 32, two to Fig. 33, one to Fig. 34, one to Fig. 35 and one to Fig. 36. They will be discussed in the order above named.

A very common form of connection is shown in Fig. 31 which consists of an angle bent into the arc of a circle. Two holes, each admitting a rod are punched in the verticle part of the angle as shown. The rod is placed in position



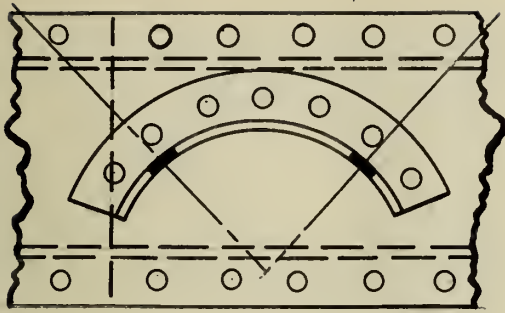


Fig. 31.

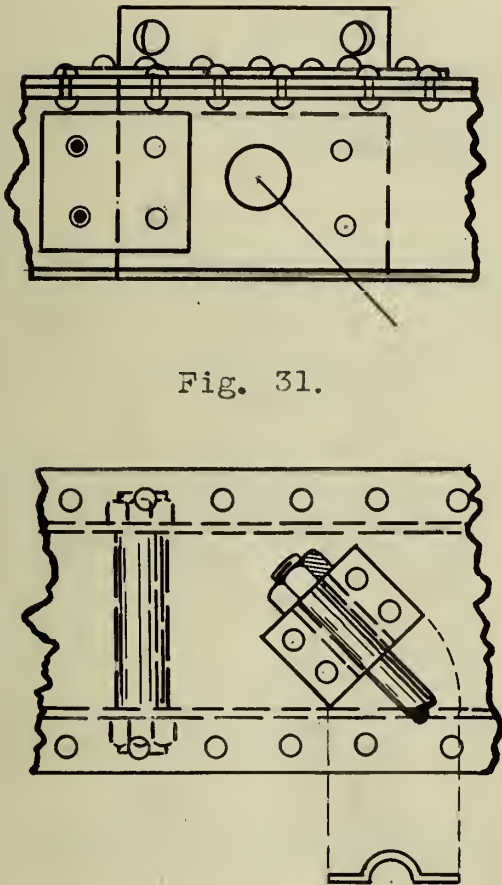


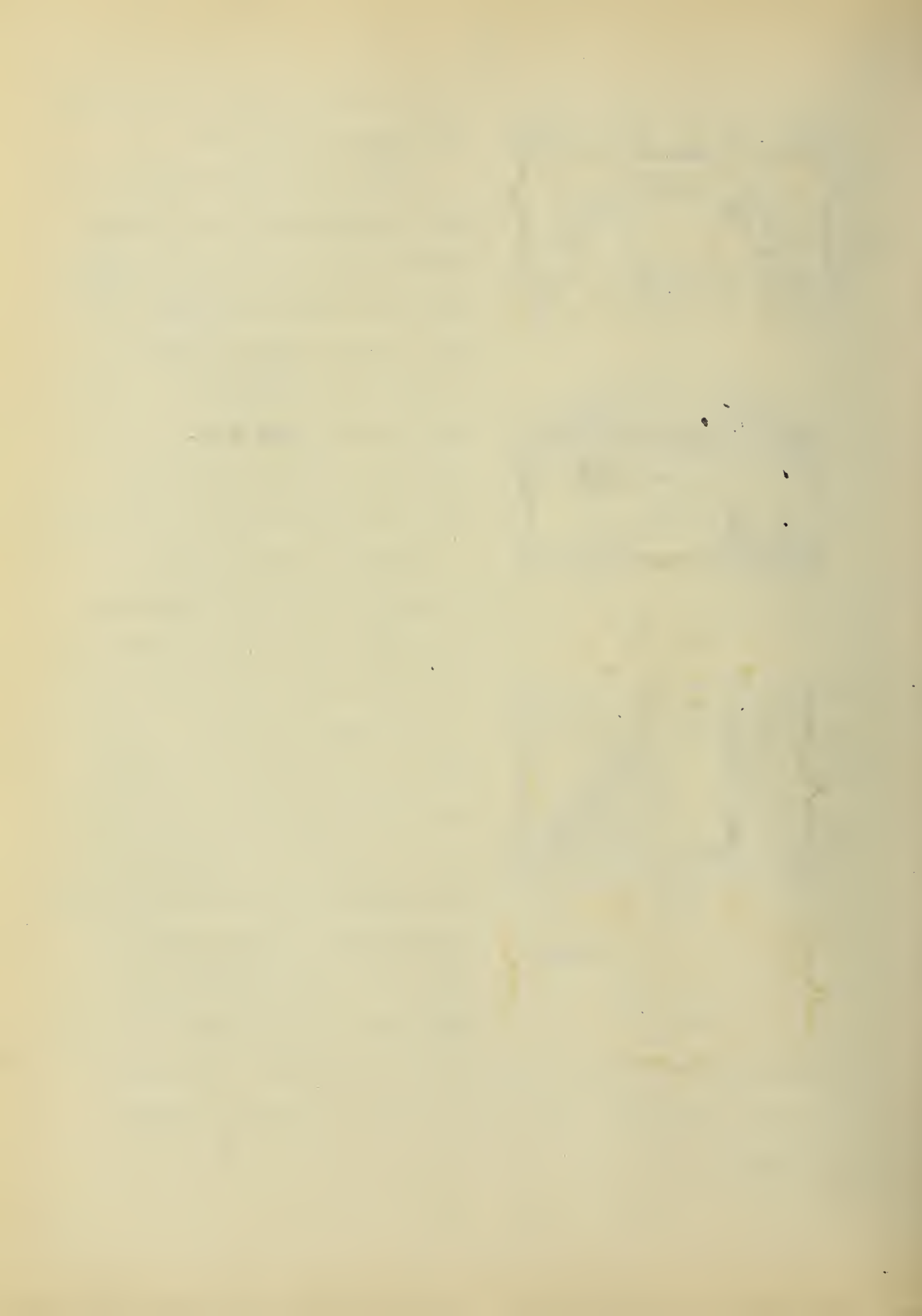
Fig. 32.

and the nut tightened to prevent the sagging of the rod.

The form is commendable in that it distributes the stress uniformly to the upper chords and may be shop riveted. Also the rods are easily placed into position and tightened up, thus facilitating erection. It is, however, a little more costly than that form shown in Fig. 2.

Fig. 32 shows a form of connection which was the most commonly used in the bridges investigated and which consists of a bent plate as shown. It also admits of facility of erection and allows the rods to be tightened by means of nuts. It does not, however, distribute the stress as uniformly as that shown in Fig. 31, since the stress is delivered on one side of the pin,

and, therefore, become eccentric in its nature. It is also somewhat expensive, this being due to the cost of bending the plate.



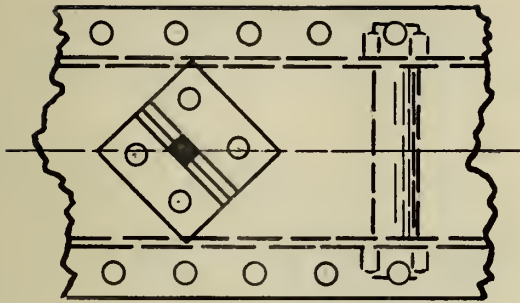


Fig. 33.

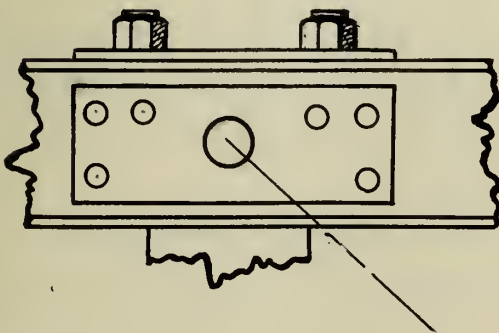
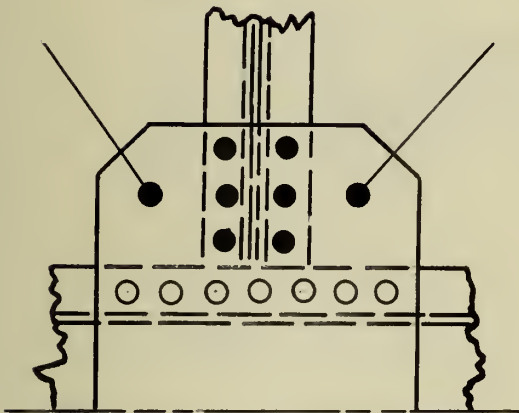
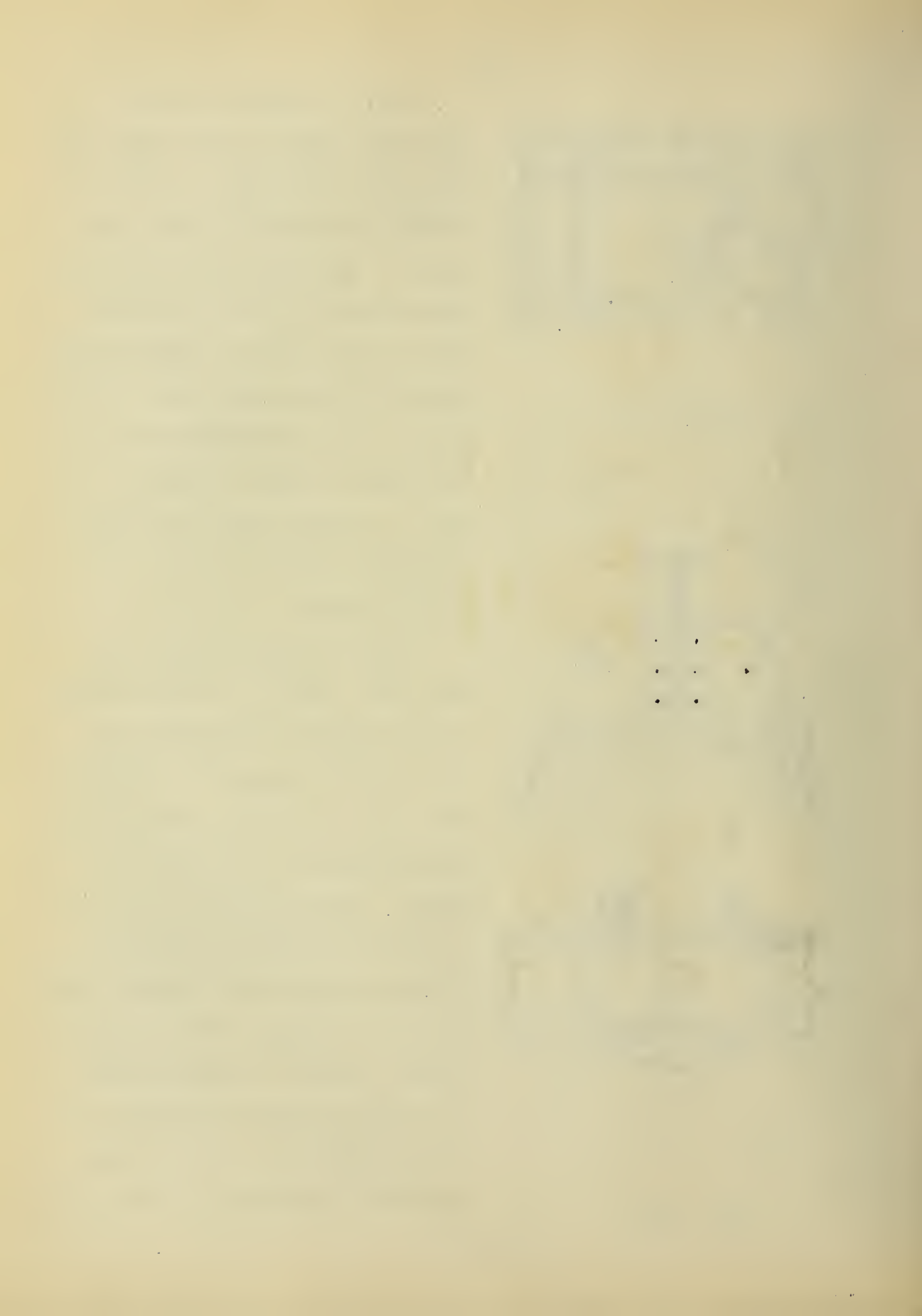


Fig. 34.

Fig. 33 shows a form of connection that is very objectionable, since the stress is not distributed uniformly to the chord member. An overturning effect is obtained, due to the rod acting with an arm equal to its distance above the horizontal plate of the angle, thus producing tension in the rivets on the far side. The form is cheaper than the preceding forms, but this adds little to its merits.

Fig. 34 shows a form that was seldom seen. It was employed by the Wrought Iron Bridge Co., on a 117-foot span of seven panels. This bridge has many unique features which are to be seen later. The use of this form of connection offers a good method of attachment of top lateral strut to chord members. The use of a forked eye-bar is objectionable on account of the increased cost of construction. It also prevents the tightening of the eye-



bar when sagging. The connection, however, is allowable and is to be preferred over that shown in Fig. 33 on account of the better distribution of stress in the chord members.

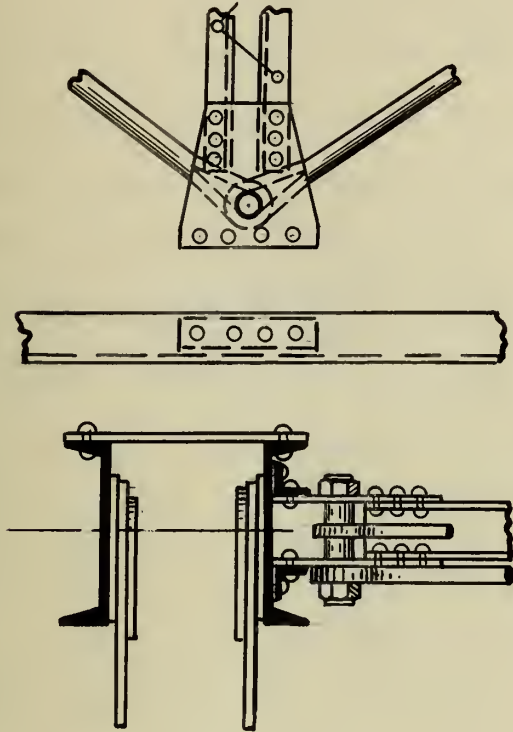


Fig. 35.

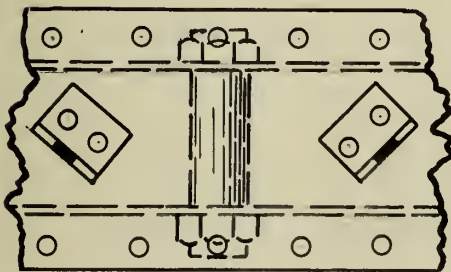
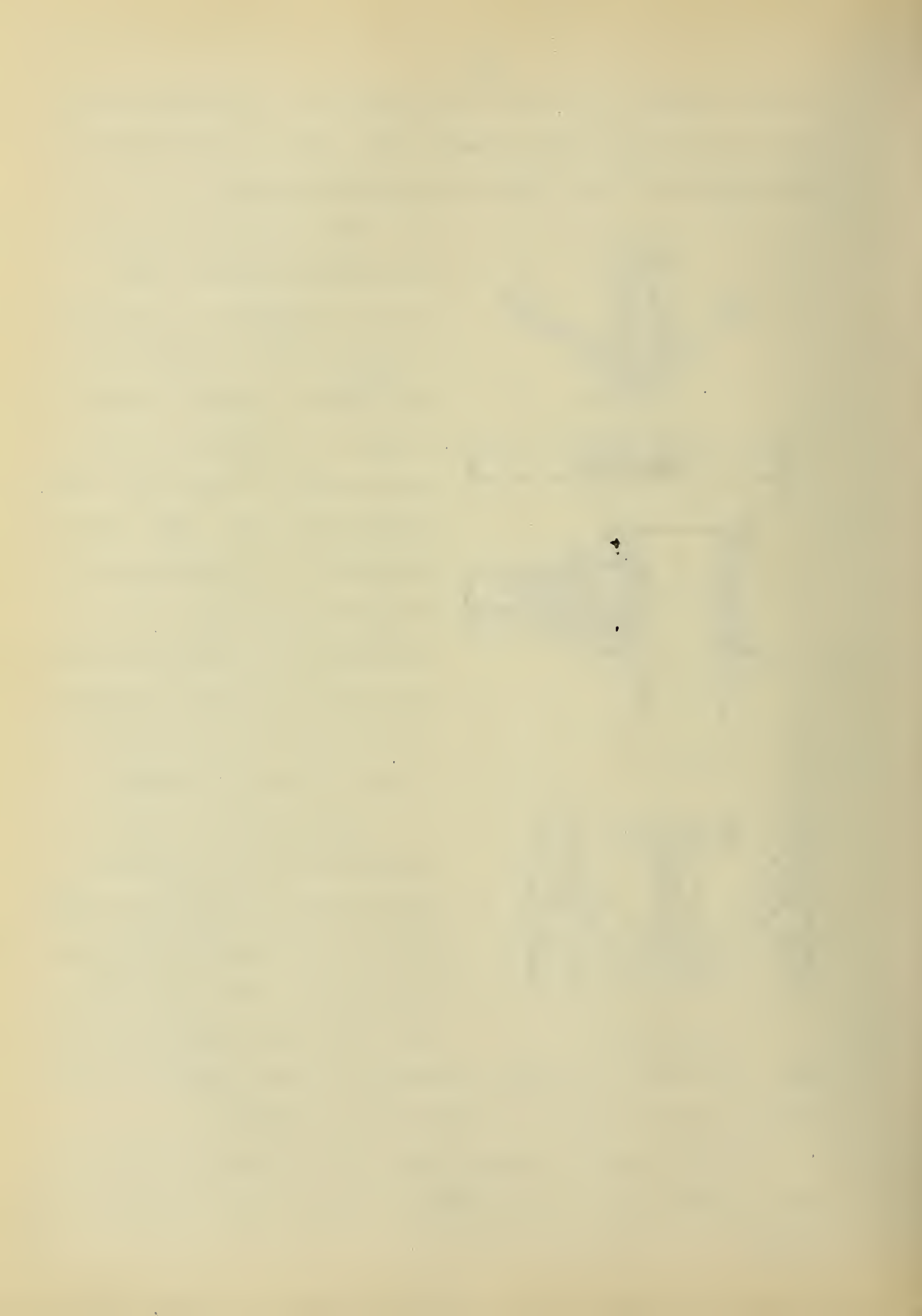


Fig. 36.

Fig. 35 shows a form of connection which was only found in one case. It was employed by the Chicago Bridge and Iron Co. in the Oakland Bridge, a bridge of 180-foot span located between Coles and Douglas Co. Ill. This form of connection is objectionable on account of its cost, its non-allowance for sagging and the fact that it causes eccentric stress in the chord member. It is not to be recommended.

Fig. 36 shows a form of connection which is very objectionable for the same reason mentioned under discussion of Fig. 33. The use of only two rivets for connection purposes should not be allowed on account of the inability to obtain the proper amount of rigidity of the system.

To sum up; for bridges where a rod is employed for diagonal purposes, types shown in Figs. 31 or 32 are to be preferred.



ART. 9. BOTTOM LATERAL DIAGONALS AND CONNECTIONS.

Cooper's 1901 specifications state that the bottom lateral system of a deck truss should be designed to meet a static wind load of one hundred and fifty (150) lbs. per linear foot, while for a through truss it should be designed to meet an additional moving load of four hundred and fifty (450) lbs. per linear foot of truss. Also, the members of the diagonal system should be designed for both tension and compression; but this specification is seldom adhered to. Therefore, for through trusses, which were the only kind investigated by the writer, the bottom lateral diagonals should be considerably stronger than the upper, and this in turn also indicates that the connections should be made stronger.

The same fundamental features required of a first class upper diagonal and its connection should also apply, and to a greater extent to a lower diagonal, for the same reasons as stated under Art. 7 p. 27. While the greater stresses to be resisted by a lower diagonal would seem to justify the use of an angle bar in order to secure the necessary amount of stiffness, it is, however, impossible to economically construct a good connection for the reason that there is a very limited available space. This condition is due to the fact that chord and pedestal plates all form a common connection at this point. Then too, an angle bar diagonal would also require expensive connections to the floor beams in order to obtain the desired rigidity of the system. And lastly, since rods which have



stood the test are in good shape, angle bars are seldom used except in riveted trusses where opportunities for good connections present themselves.

There are two types of lower lateral connection in a pin connected bridge, one type being seen at each end of the span, the other type being seen at the connections of the lower diagonals to the floor beams at the intermediate panel points. They will be discussed in the above order for the pin connected trusses, after which the same order will be followed out in the discussion of the lower lateral systems of riveted trusses.

A very good connection has not as yet been devised for the connection of the lower lateral system to the end of the bridge.

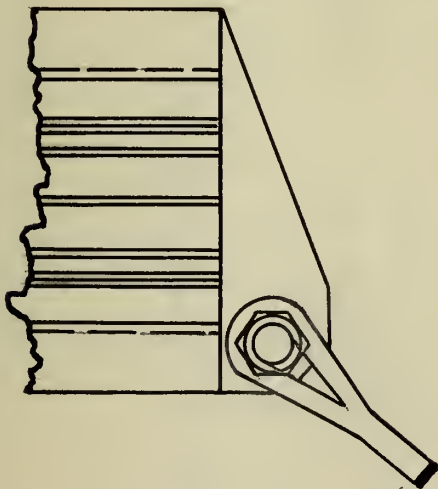


Fig. 37.



Fig. 38.

Fig. 37 shows a type of connection which was found in eight cases of the bridges investigated. It is an economical type, but does not distribute the stress uniformly into the plate. It is to be recommended on account of its cheapness, for average-sized bridges of spans of not more than 150 feet, since it requires no bent plates or forked rods.

Fig. 38 shows a type of connection that was found in



nine cases, showing its use to be about the same as that for the form shown in Fig. 37. It is identical with the preceding form except that the rod is forked, thus making the connection more expensive but distributing the stress to the base plate

more uniformly. It could, therefore, be used for longer spans than those indicated for the connection of Fig. 7.

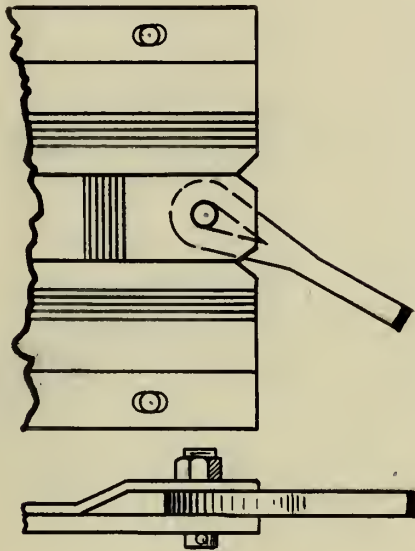


Fig. 39.

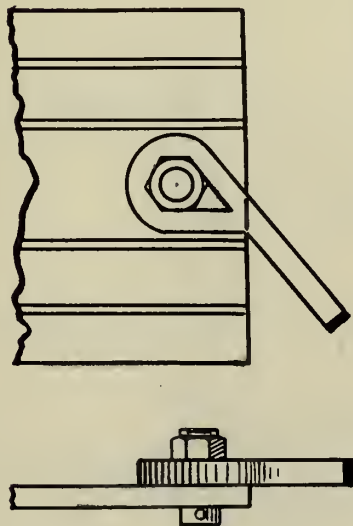


Fig. 40.

Fig. 39 shows the most common form of connection employed in the bridges investigated, fifty-six types of this kind being found. It is an economical form, being less expensive than that shown in Fig. 38 although not distributing the stress as evenly to the base plate. Its use is to be recommended for all bridges, since it is about as good a type as can be devised, and still come within the limits of expense.

Fig. 40 shows a type of connection embodying the same principal as that shown in Fig. 37. It does not, however,



distribute the stress uniformly on account of the peculiar form of the diagonal and the lack of a fastening at the top of the bolt. The bent head of the diagonal makes it expensive, and it is not to be recommended. Only three cases of its use were found.

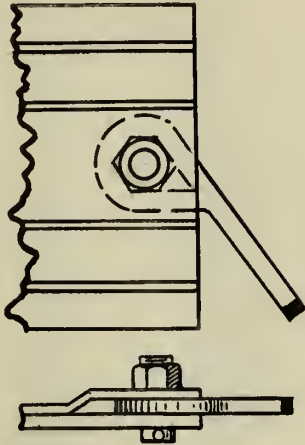


Fig. 41.

Fig. 41 shows a form identical with the preceding except that a bent plate is used. While the use of a bent plate makes it more expensive, the more uniform distribution of stress to the base plate makes it the more desirable.

Only one case of this form was

found.

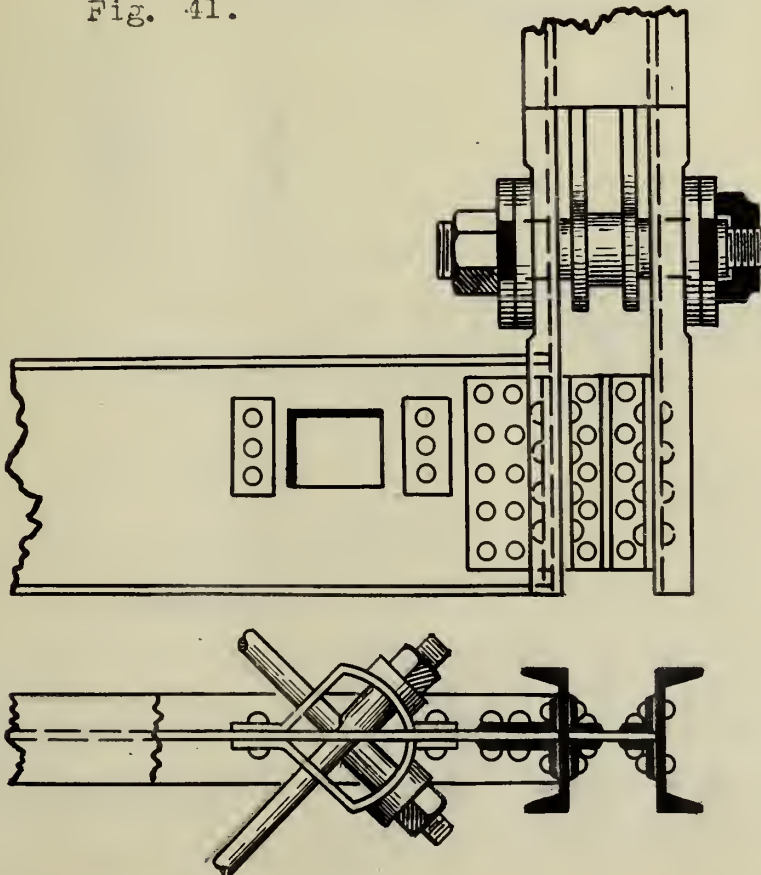


Fig. 42.

A very common and efficient connection is represented in Fig. 42. The detail is very good in that there are no rivets in tension and the stress is distributed fairly uniformly



to the floor beam. The form is, however, somewhat expensive as both the cutting of the web of the floor beam and the use of bent plates involves an increase of cost. The type is, however, on account of its efficiency recommended for all spans.

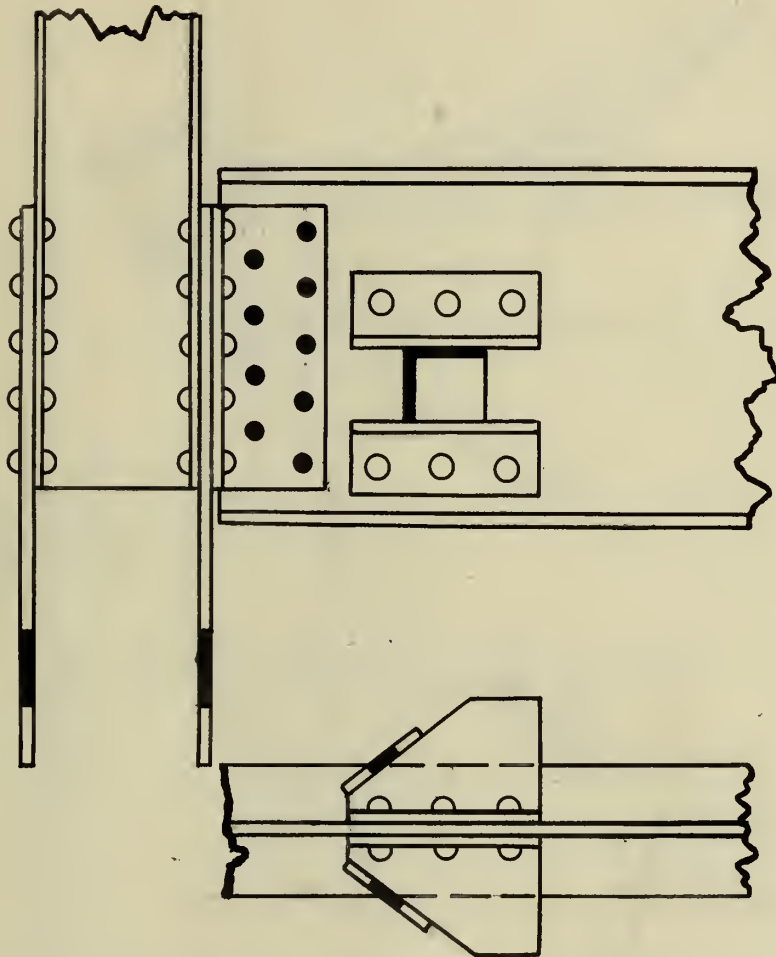
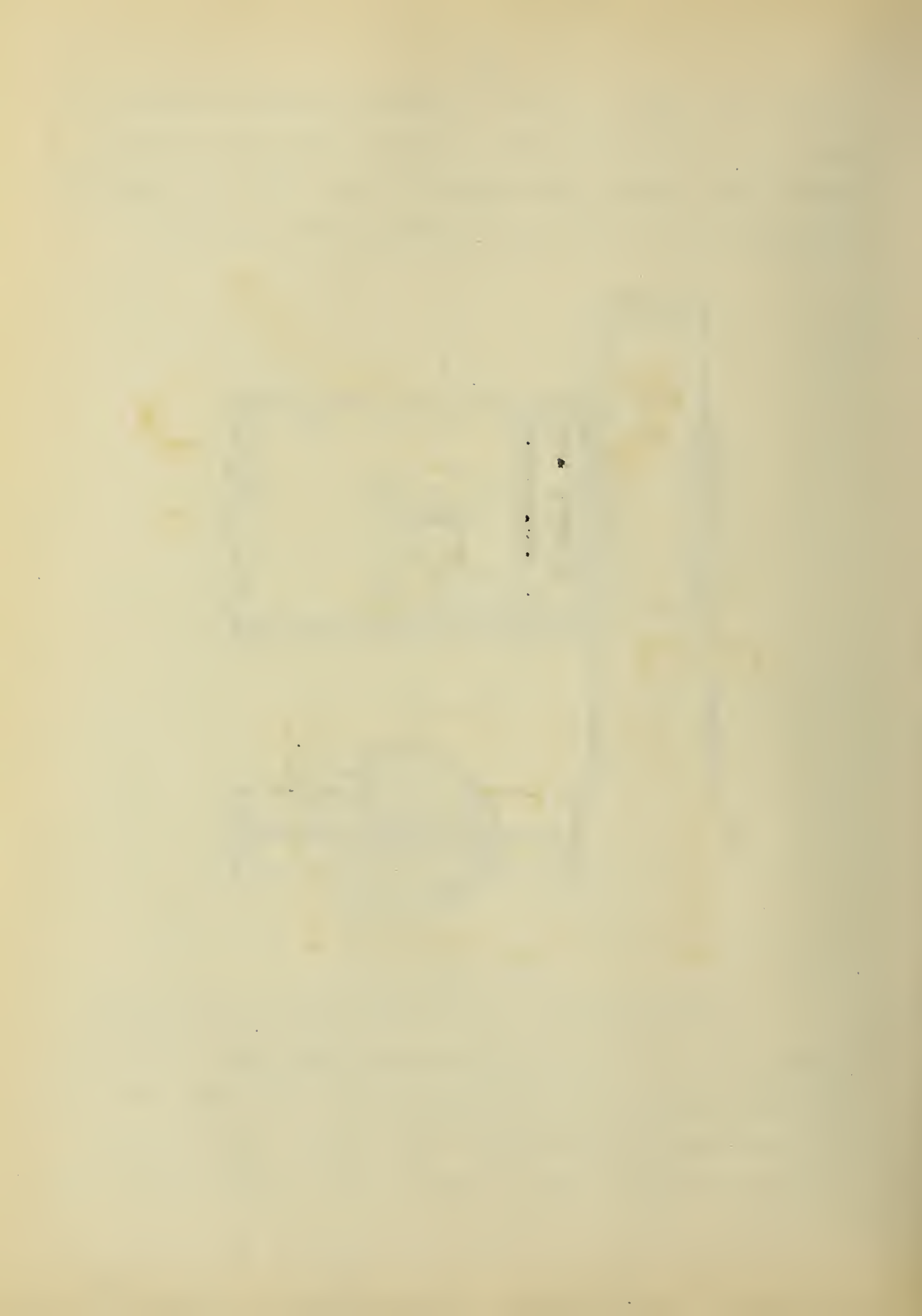


Fig. 43.

A more economical and efficient detail is to be seen in Fig. 43. It is more efficient than that shown in Fig. 42 in that the stress from the laterals is distributed more evenly to the floor beam on account of the greater area of contact offered through the use of angles instead of bent plates. The



use of angles also decreases the cost as they are not so expensive as the bent plates. As in the preceding case there are no rivets in tension. This type, therefore, is also to be recommended, being preferable to that shown in Fig. 42.

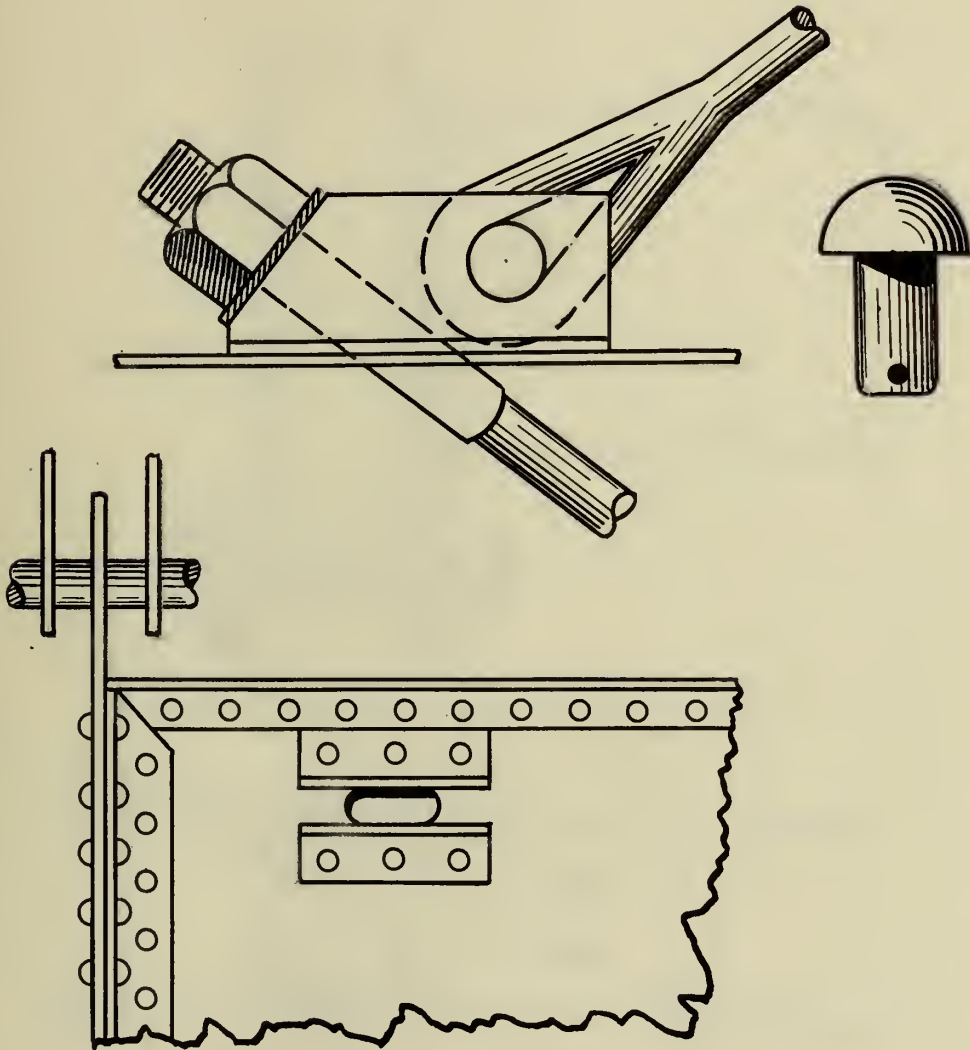


Fig. 44.

Fig. 44 shows an uncommon and inefficient method of connection of lower lateral diagonals to floor beam as the stress in member "b" is always zero when "a" is acting tension in rivets will result. The connection of rod "b" to the angle at "c" is faulty in that the bending moment is developed in the



pin at that point and stress is not transmitted evenly to the angles. This fault might, however, be remedied by the use of a forked rod but this would entail extra expense. The form is, therefore, not to be recommended.

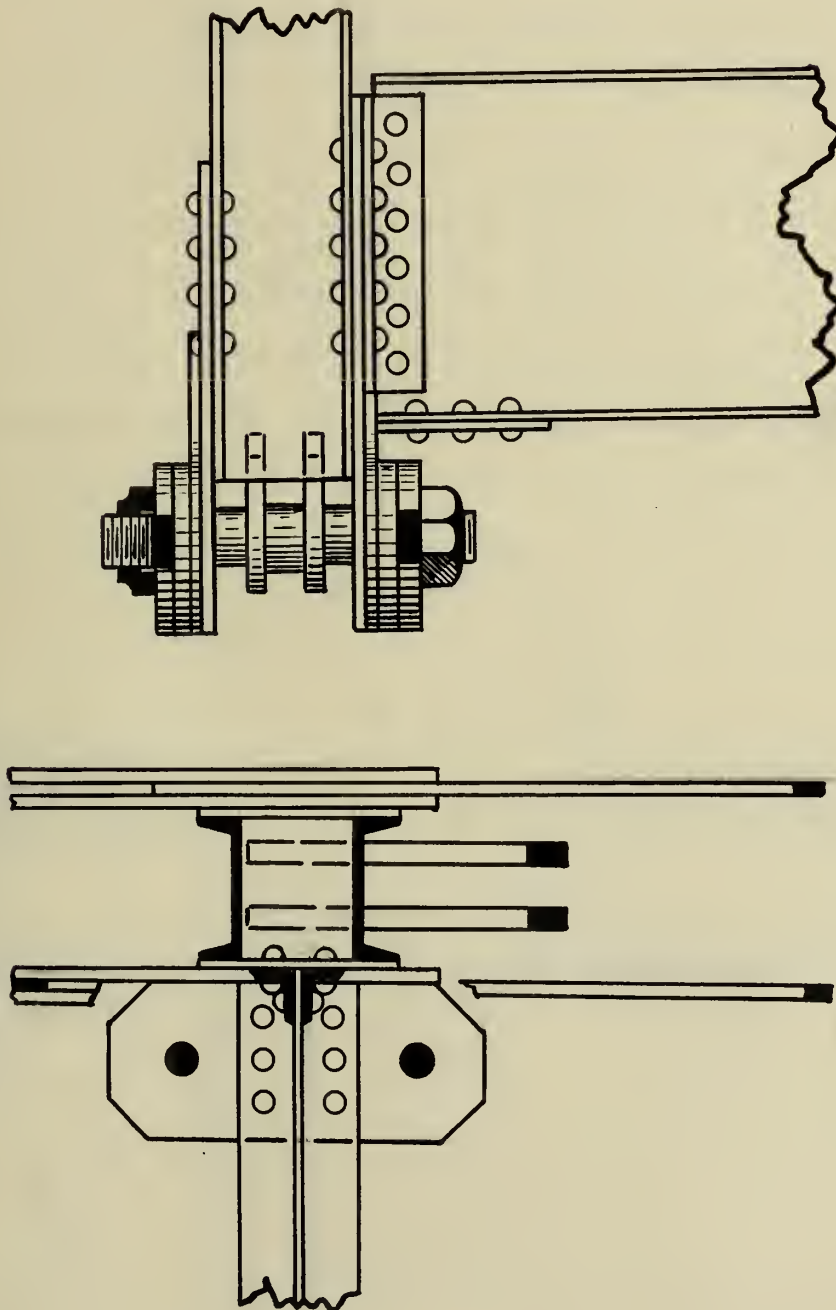


Fig. 45.



A flimsy and inefficient connection is shown in Fig. 45, where a solitary plate is the only thing required to form a connection of the lower lateral system to the floor beam. The connection is faulty in that eccentric stresses are developed in the rivets since only one of the laterals acts at one time. Then too, the use of a forked bar becomes necessary which involves extra expense.

It is inefficient in that a bending moment is developed in the pin which in turn does not distribute the stress evenly to the plate. The plate, therefore, cannot transmit the stress uniformly to the floor beam. The detail is, however, certainly very economical and is on this account to be recommended for spans of less than 70 feet, where no very large stresses obtain in the lower lateral systems.

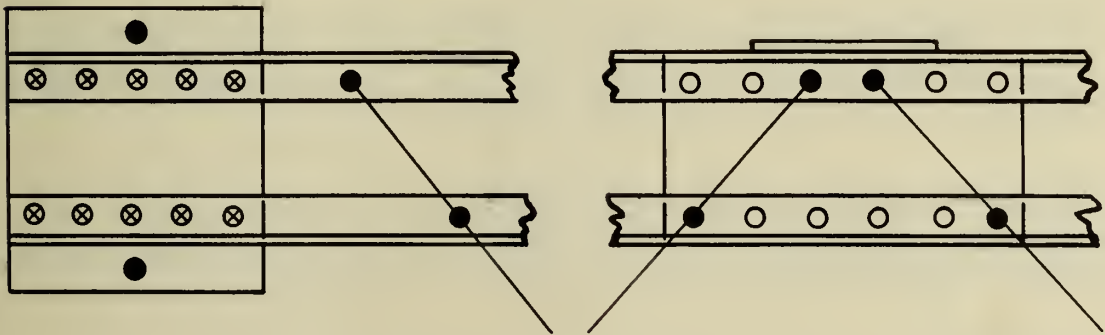
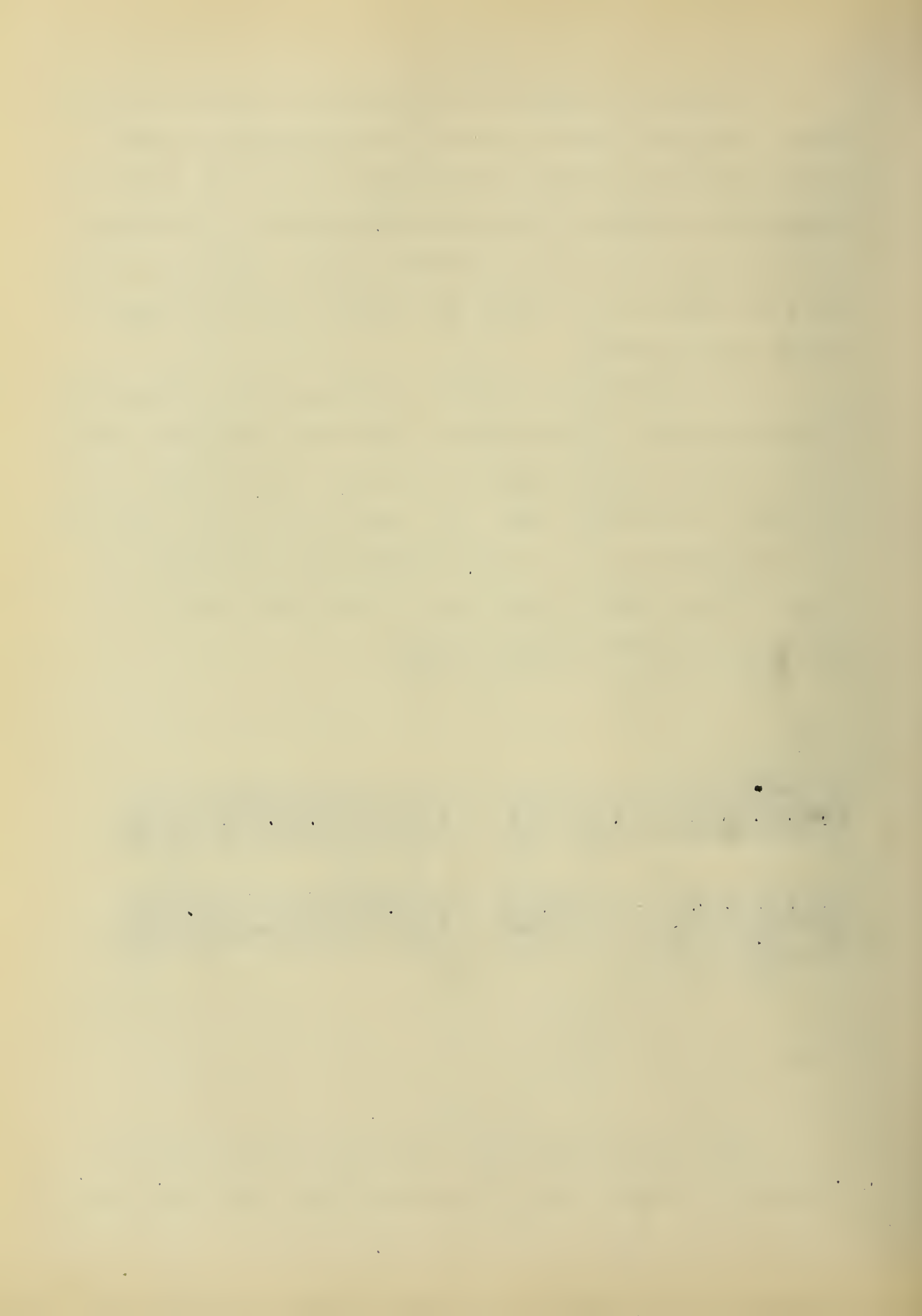


Fig. 46.

Fig. 46 shows the lower lateral connections which are employed in a small span riveted pony truss, the lateral system



in this case being connected to the lower chord instead of to the floor beam as is always the case in pin connected trusses. The detail is efficient in that no tension in rivets is permitted and sufficient riveting area is offered. It is economical in that only one plate is required. The detail is, therefore, recommended where lower chords and diagonals are made up of angles.

#### ART. 10. PORTALS AND SWAY BRACINGS.

A first class portal should possess the following features:-

- 1st. It should be economical in section.
- 2nd. It should possess sufficient rigidity and strength.
- 3rd. It should offer an efficient and economical connection to the end post.
- 4th. Field riveting should be a minimum.
- 5th. It should be of such form as to reduce bending moment of wind on the end post to a minimum and at the same time give sufficient head-room.

It should be economical in section in that it should require a minimum amount of material to take the stresses required, thereby usually reducing the cost to a minimum.

It should possess sufficient rigidity in order to keep the distance between the end posts and upper chords constant, thereby preventing any eccentric stresses which might arise. All parts should have the required strength, and there should not be bent plates to tend to weaken the structure.



The connection of the portal strut to the end post should be efficient in order that none of the rigidity developed in the portal be lost. If the connection is not efficient, rivets may be subjected to tension, eccentric stresses may be developed in end posts, and various other troubles may result, thereby reducing in a great extent the efficiency of the portal. A poor connection to a good portal greatly reduces the efficiency of the latter. Especially is this true of the connection at the foot of the knee brace where the bending moment of the end post is a maximum. While a connection should be efficient, it should also be economical, for an expensive connection employed for an inexpensive portal is a waste of money.

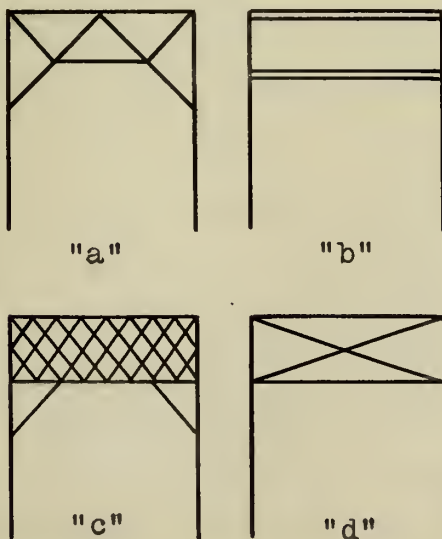
A good portal should require as little field riveting as possible; for rivets that are riveted in the field are not considered as efficient as those riveted in the shop, and therefore more rivets, material, and labor are required to put portal in place, thereby increasing the cost.

The portal should be of such form as to reduce the bending moment of wind on the end post to a minimum, and still allow sufficient head-room. For bridges of long spans the height which is usually made about one sixth of the span, will be sufficient to give ample headroom, and at the same time allow the portal to be fastened proportionally, much lower than on short span bridges.

For smaller spans, however, great care must be taken that while locating the foot of the knee brace as low as possible on the end post, thereby reducing the bending moment



of the wind which is always a maximum at this point, sufficient head-room be left. Care too must be taken in the locating of the portal knee brace as low as possible on the end post that the greater stress developed, and consequently the added cost for the extra material required does not more than balance the added cost of the larger channels required in the end posts when the knee brace is placed higher up. It would hardly be possible to deduce an exact mathematical formula for the determination of the most economical angle for the knee brace to make with the end post as there are so many varying conditions to be taken into account, and so the designer must depend upon his judgment. It does not, however, make so much difference for end posts are usually made amply large to resist any bending moment that may be brought to bear upon them provided the knee brace is located at least one sixth of their length down. The common practice for single paneled portals is to have the knee brace make an angle of forty-five degrees with the end post.



There are many types of portals employed in highway bridges. Some of the most common forms are shown in Fig. 47, "a" being the commonest. Most designers agree that for bridges of ordinary spans up to 130 feet, lacing is not necessary and is too

Fig. 47



expensive in as much as a great amount of material is required, and the shop cost runs quite high. However, where great rigidity is required lacing should be used. When lacing is employed it should be double since it is required to resist the shear. Double lacing should always be riveted at the intersections so as to obtain the greatest amount of rigidity.

Cooper's 1901 Specifications state that "the distance of the center of the rivet from the edge of any of the connecting plates should never be less than 1-1/4 inches."

The use of top lateral struts to serve as portals is not to be recommended for bridges of spans of over eighty feet as too large a bending moment due to wind can be developed in the end post which must, therefore, be made abnormally large to resist the large flexural stress which may be developed in that case.

Very often the large bent plates which serve as connections for the top lateral diagonals to the portal and end post are riveted to the portal in the shops. Especially is this true for the most common type of portals which is shown in "a", Fig. 47. This is not to be recommended for the bent plate is very liable to become distorted during transportation, is expensive, and does not evenly transmit the stress from the diagonal to the end post and portal.

Portals are not required for spans of less than 50 feet where pony trusses may be used.

The connection of the portal strut to the end post is usually made by means of a connecting plate which is fastened



to either one or both of the upper flanges of the channels forming the end post. The connecting plate in that case should always extend clear across the end post so as to distribute the stress more evenly, and offer a more rigid connection than if the plate was only fastened to one side. Where double lacing is employed in place of a cover plate, one lacing bar may be placed over the plate with the other one beneath it, thereby placing the connecting plate at a distance from the channel equal to the thickness of the lacing bar. For single lacing, the lacing bar is always placed on top. It would, however, be better if the lower lacing bar were dispensed with entirely, thereby allowing the connecting plate to fill its place. The usual method however is to run the lacing, single or double whichever it may be, up to the connecting plate.

Sometimes the portal strut is connected to the end post by means of connection angles which form a connection between the angles of the portal strut and the web of the nearest channel of the end post.

A discussion of some of the commonest types of portals with their connections will now be taken up.

The form shown in Fig. 48 represents the most common type of portal found by the writer, sixty-four of the bridges investigated having portals of this type. Fig. 49 represents a form of a heavier kind of this type which was found on the Plato Bridge, a bridge of 273-foot span, built by the Massillon Bridge Co. over the Iroquois River. This type is, of course, subject to many modifications; this being especially



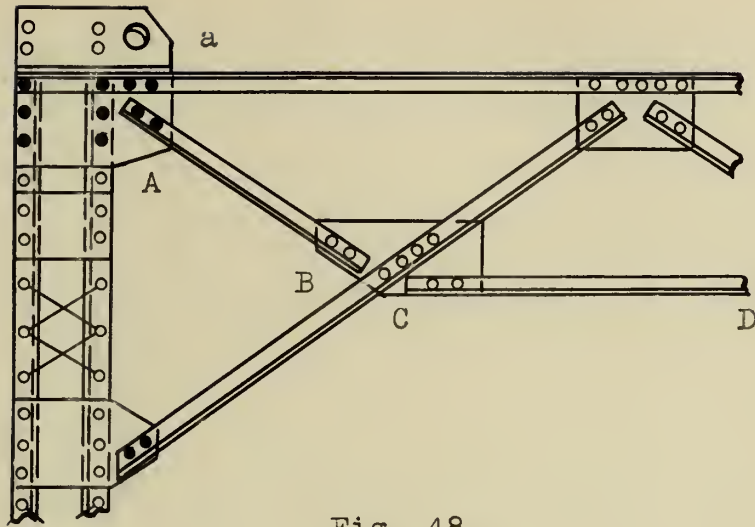


Fig. 48.

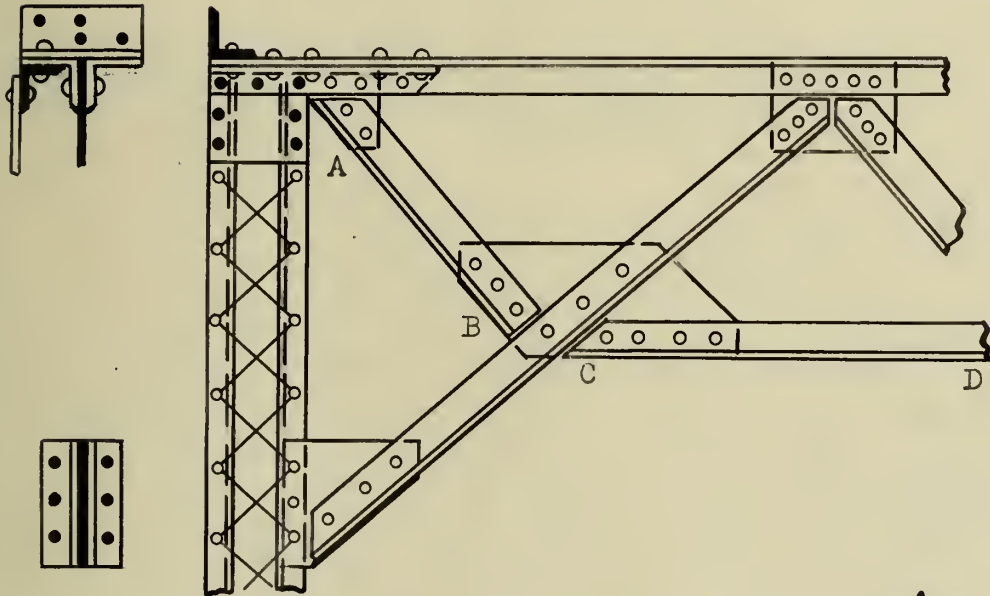
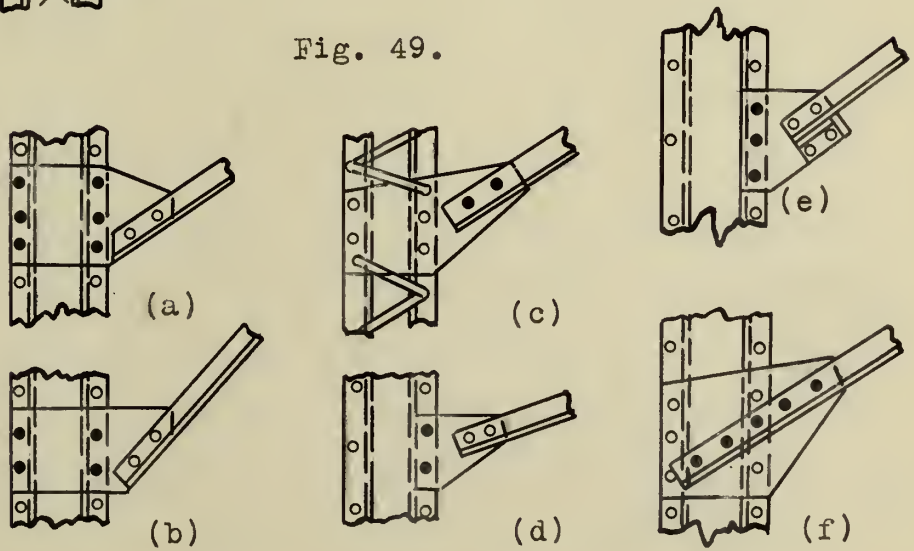


Fig. 49.



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true as regards the various forms of connecting plates used. Almost every portal of this type has a different form of connecting plate at the knee brace. Some of the commonest forms are shown in Fig. 50. The form of connection shown at Fig. 50 is undoubtedly the best distribution of stresses from the portal to the end post on account of the great area allowed for rivet connection.

This form of portal is to be recommended for spans up to 150 feet in length, except where broken chord trusses are used, when it may be used for longer spans. An example of its use in a longer span is seen in the Plato Bridge, the portal of which is shown in Fig. 49.

It is to be recommended for the following reasons,

- 1st. It is simple and, therefore, economical.
- 2nd. It is sufficiently strong and rigid, since two angles riveted together can, if they be of the larger common sizes employed, easily resist the stresses developed in portals of bridges of spans up to 150 feet in length, since those stresses seldom exceed thirty thousand pounds. Members AB and CD take practically no stress, and serve to keep the portals rigid.
- 3rd. The connections of the portal at the knee brace to the end post is both economical and efficient, since but little material is required to make the connection, and sufficient area for attachment of rivets is offered.
- 4th. There is but very little field riveting required.

The attachment of the bent plate at "a" Fig. 48 should not be allowed and is not economical for reasons mentioned on



p. 44. The method of connection of the top lateral diagonals to the upper chord employed on the other parts of the bridge, should be used instead.

Fig. 51, which is shown below, illustrates a form of portal strut that was found in but one case. In another case,

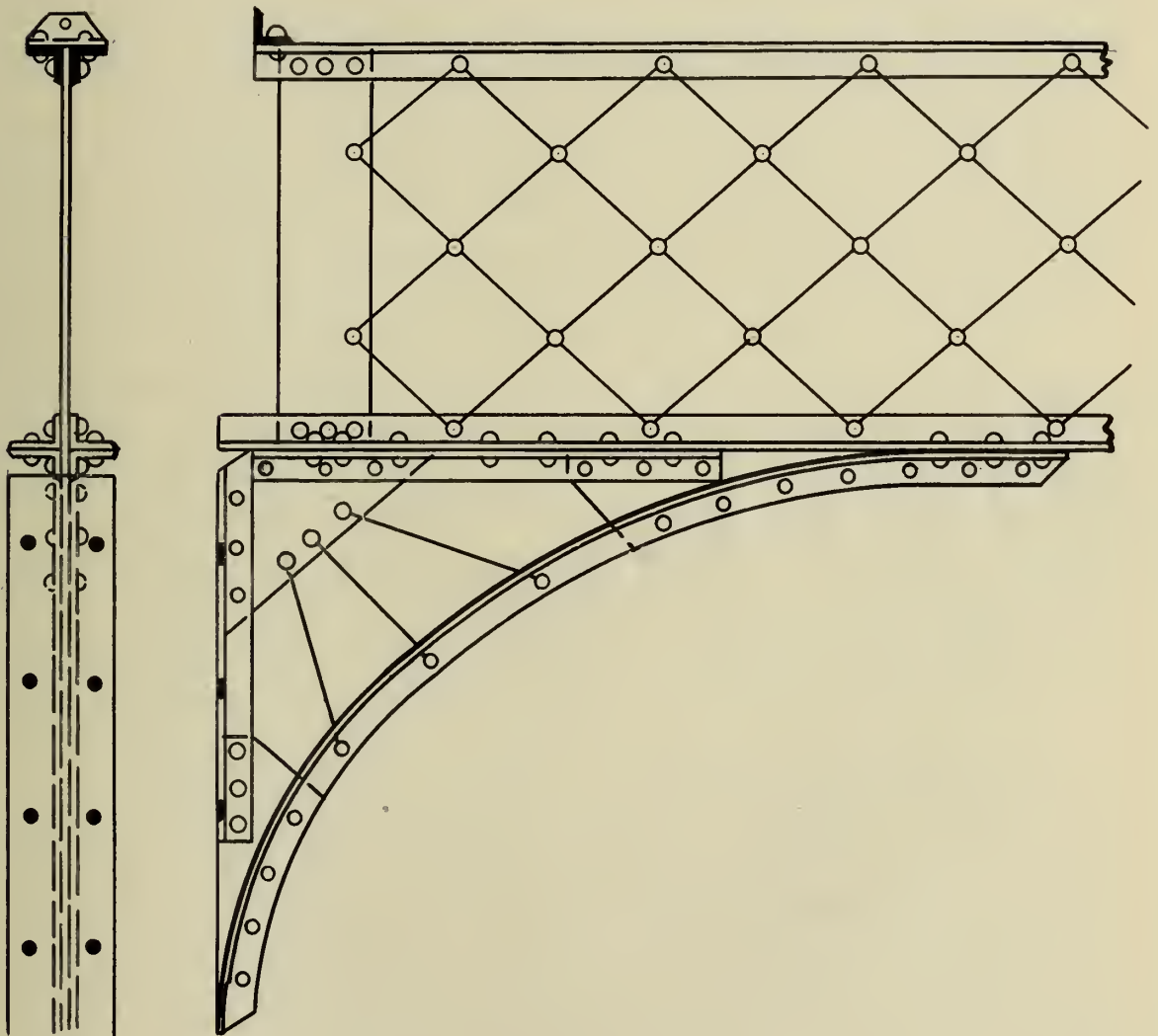


Fig. 51.

the form employed was the same with the exception that three



lacing bar plates at "a" were missing. In both cases the spans of the bridges, where in the above types were found employed, were 120 and 105 feet respectively. This type is very rigid on account of the amount of lacing employed; but it should not be employed for spans ranging from 100 to 150 feet in length on account of its excessive cost. The cost of the bent angle is large and its rigidity is not nearly as great as that of the latticed part above. The connection is also expensive since it requires two angles, but is at the same time very efficient for those angles are offered a rigid method of connection to the channels of the end post as is seen in Fig. 51. The form would be improved if the latticed part were deeper, thereby making the circular bent angle of shorter radius, and consequently stiffer.

This type of portal should not be used on bridges having spans of over 150 feet in length, since for bridges of less span it is uneconomical, and for reasons stated in the above discussion, is not to be recommended.

A form similar to that in Fig. 51 but improved as suggested in the preceding discussion is shown in Fig. 52. This form was found in one bridge, the bridge having a span of 166 feet and built by the Massillon Bridge Co. over the Maumee River at Waterville Ohio. The form is to be recommended for all bridges of spans greater than 150 feet. Its rigidity and strength are increased by the angles which are used instead of lacing bars, and also by the way in which they are held in place by the small plates at each intersection. It



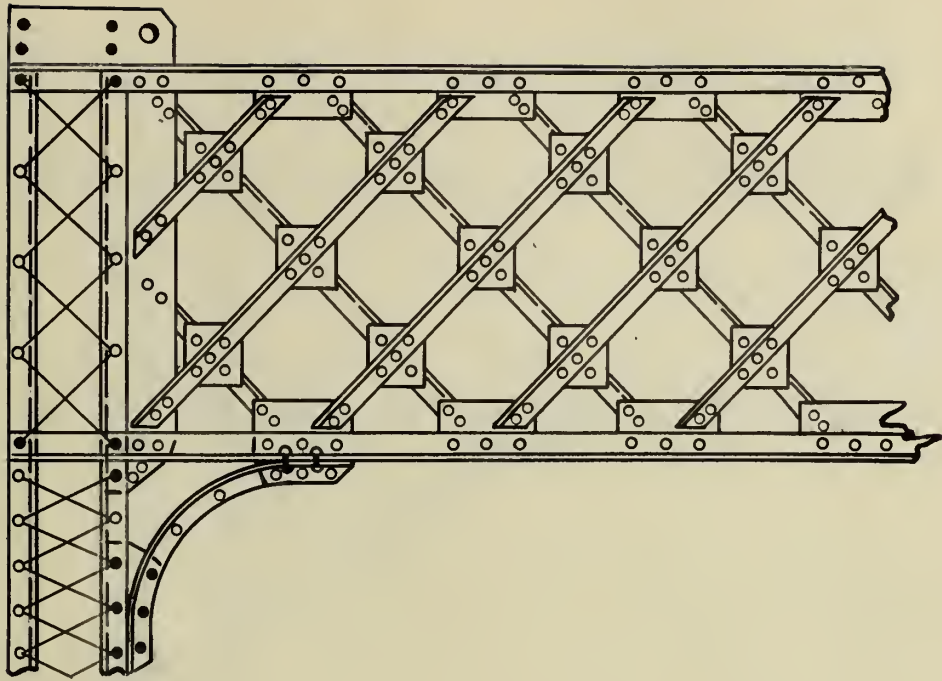


Fig. 52.

is also to be noticed that the laced part is very much deeper than the arch of the circular bent plate, thereby making the entire form of greater uniform rigidity than that shown in Fig. 42, for reasons mentioned in the discussion of that figure.

The form of portal strut shown in Fig. 53 was only found in two cases. It is only to be allowed for spans varying from 130 to 150 feet in length. The form is somewhat expensive due to the cost of latticing, but is quite rigid and serves its purpose well. It has one undesirable feature that prevents its being recommended for bridges of longer span; the method of connection to the end post at the foot of the knee brace is poor. The stress is only distributed evenly to the end post on account of the method of fastening, and the small number of rivets used prevent any great rigidity being developed in the



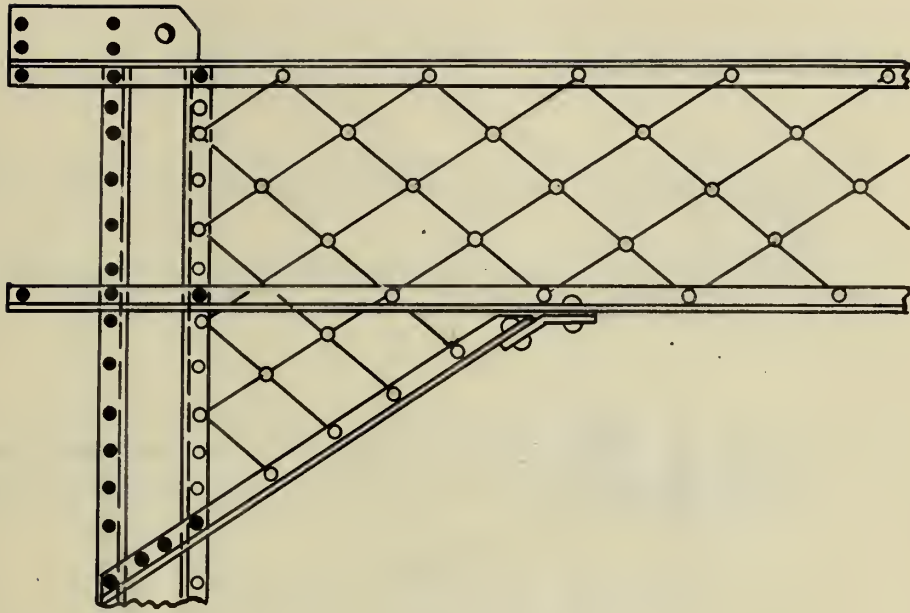


Fig. 53.

connection. It also decreases the efficiency of the great rigidity developed in the latticed part above. It is, therefore, the opinion of the writer that, for the spans for which this type is applicable, the form shown in Fig. 32 be used instead. A modification of the type may be seen when the lower horizontal angles do not extend clear across but end with the angles forming the knee brace. This, however, is little of an improvement over the preceding form, the only improvement, if any, being that it lessens the cost a little. This detail was found in but one case.

Fig. 54 shows the form of portal strut employed in the Oakland Bridge of 180-foot span, built by the Chicago Bridge and Iron Co. over the Iroquois River. It is not to be rec-



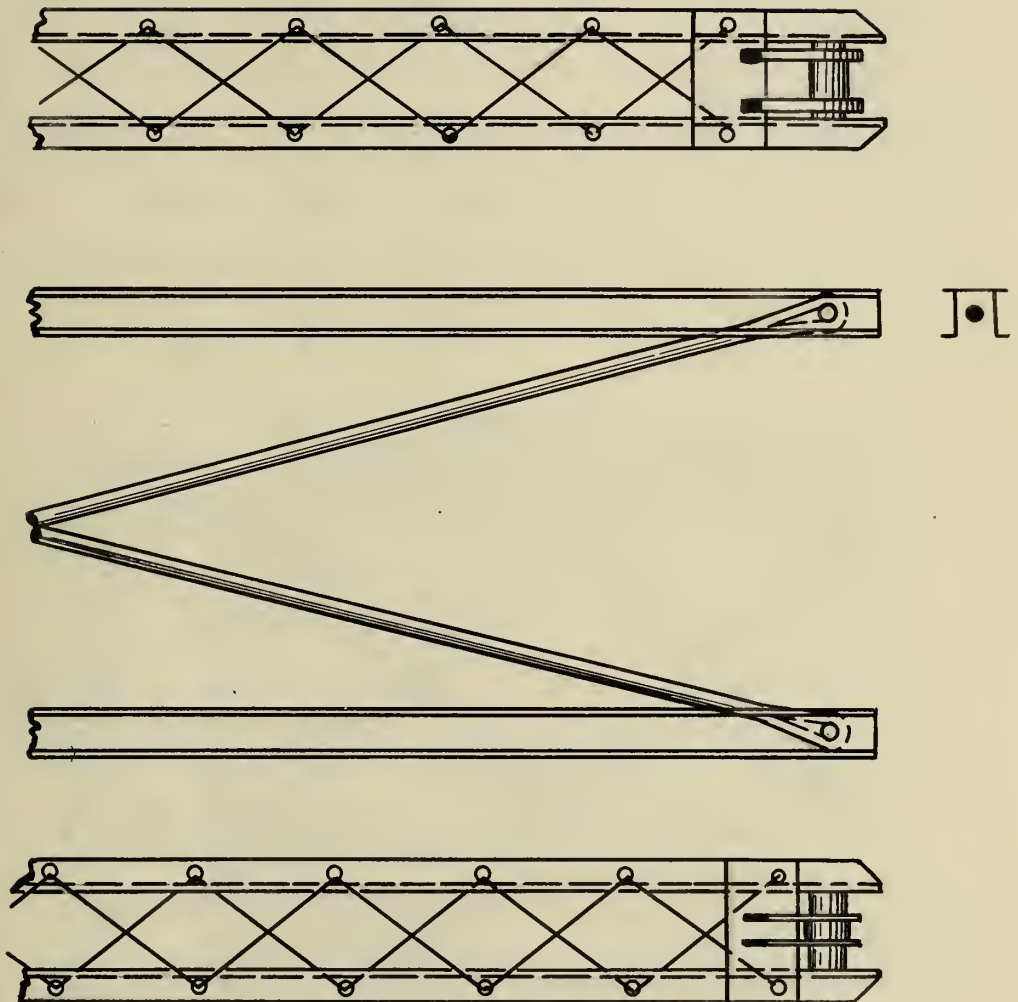


Fig. 54.



ommended on account of the excessive cost and the lack of rigidity. The two loop bars cannot give a rigid connection and therefore the portal taken as a whole cannot be said to be rigid, however capable of resisting stress that it may be. It is expensive on account of the great amount of shop work required to build up the strut. Then too, the connections to the end posts offered are poor since but little area of attachment is offered to the small connecting plates that support the pins. The form shown in Fig. 36, p. 32 is on account of its greater rigidity to be recommended instead of this type.

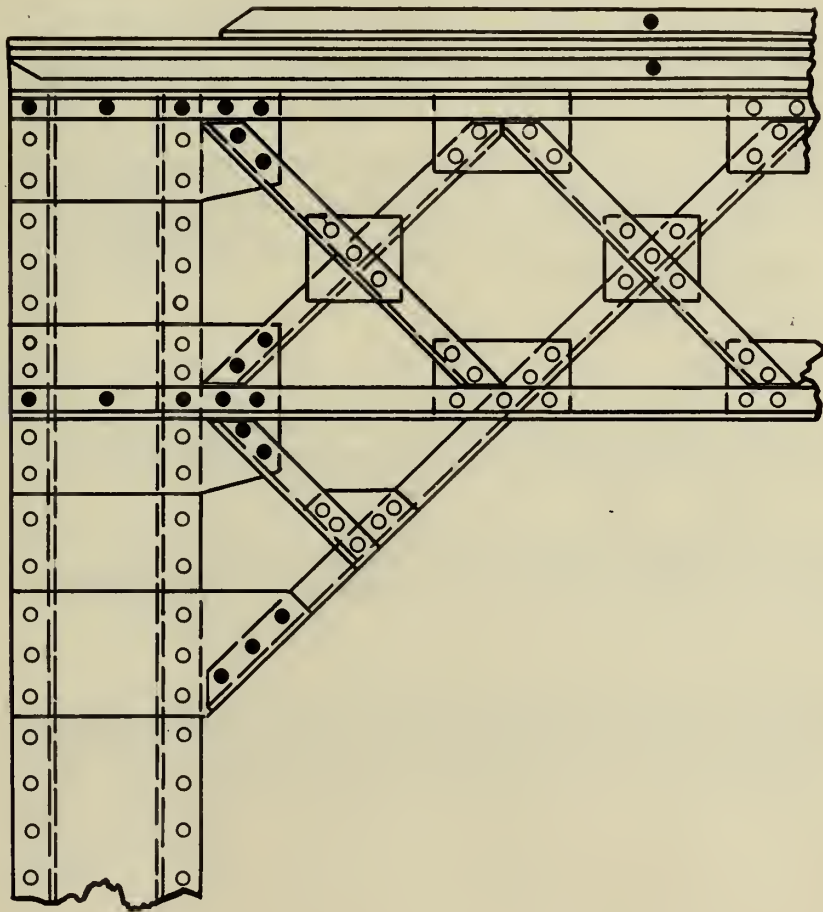


Fig. 55.



Fig. 55 shows a form of portal strut that was found employed in a 105-foot span bridge located at Chesterville, Douglas Co., Ill., and built by the Chicago Bridge and Iron Co. The form is very efficient and rigid on account of the angles being used in place of lacing bars, and the use of the small connecting plates at each of the intersections. The connection at the foot of the knee brace is also rigid and strong on account of the area of connection over which the stress may be distributed, and the number of rivets which are employed. This type, however, is too expensive for a bridge of 105-foot span. The portal shown in Fig. 48 would, for 105-foot span bridge, serve the purpose as well and be less expensive.

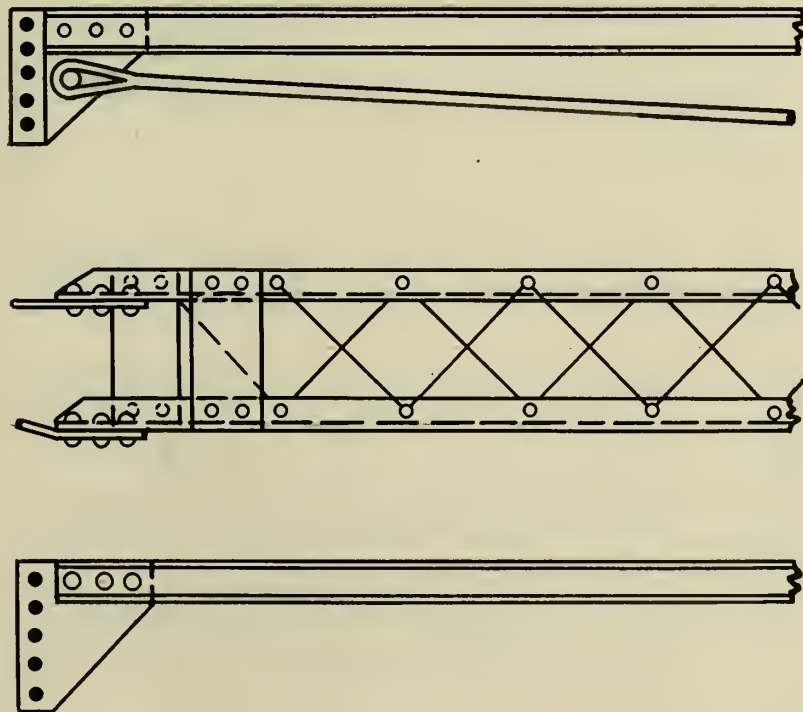


Fig. 56.



Fig. 56 shows a form of portal that was found in a bridge of 160-foot span and built by the Indiana Bridge Co., at Mahomet Ill. The form is not as rigid as the latticed forms. It is also expensive on account of the large shop cost. It is also expensive on account of the jaw eye bars which are used. Then too, its mode of connection to the end post is not good on account of the fact that the connecting plate is only fastened to one of the channels instead of to two as suggested on p. 45 . Therefore, the type is not recommended.

The same requisites of a good portal also apply to sway bracing. The use of sway bracing is not necessary for highway bridges of less than 150-foot spans, since good top lateral struts are capable of taking the stresses in top lateral systems. If intermediate posts are not greater than 20 feet apart, spans of greater length may be made to do without sway bracing; but it is not policy to increase the number of posts in order

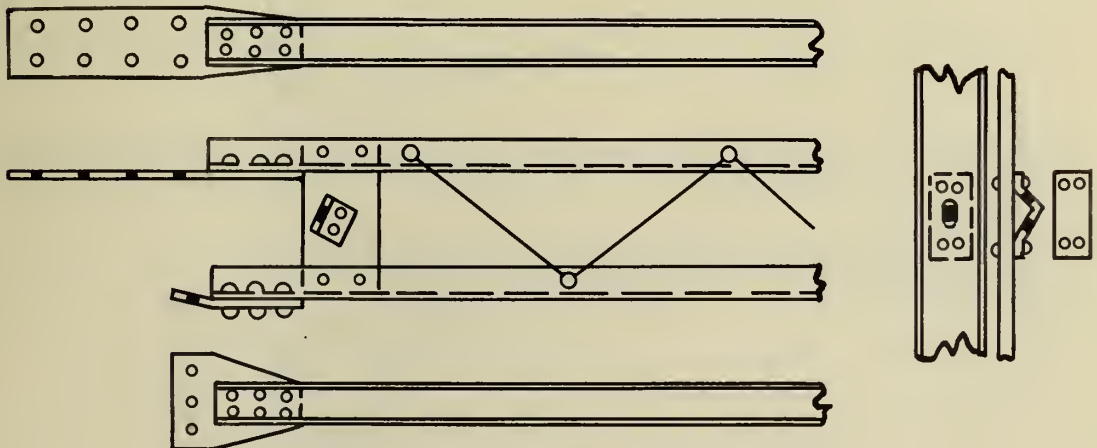
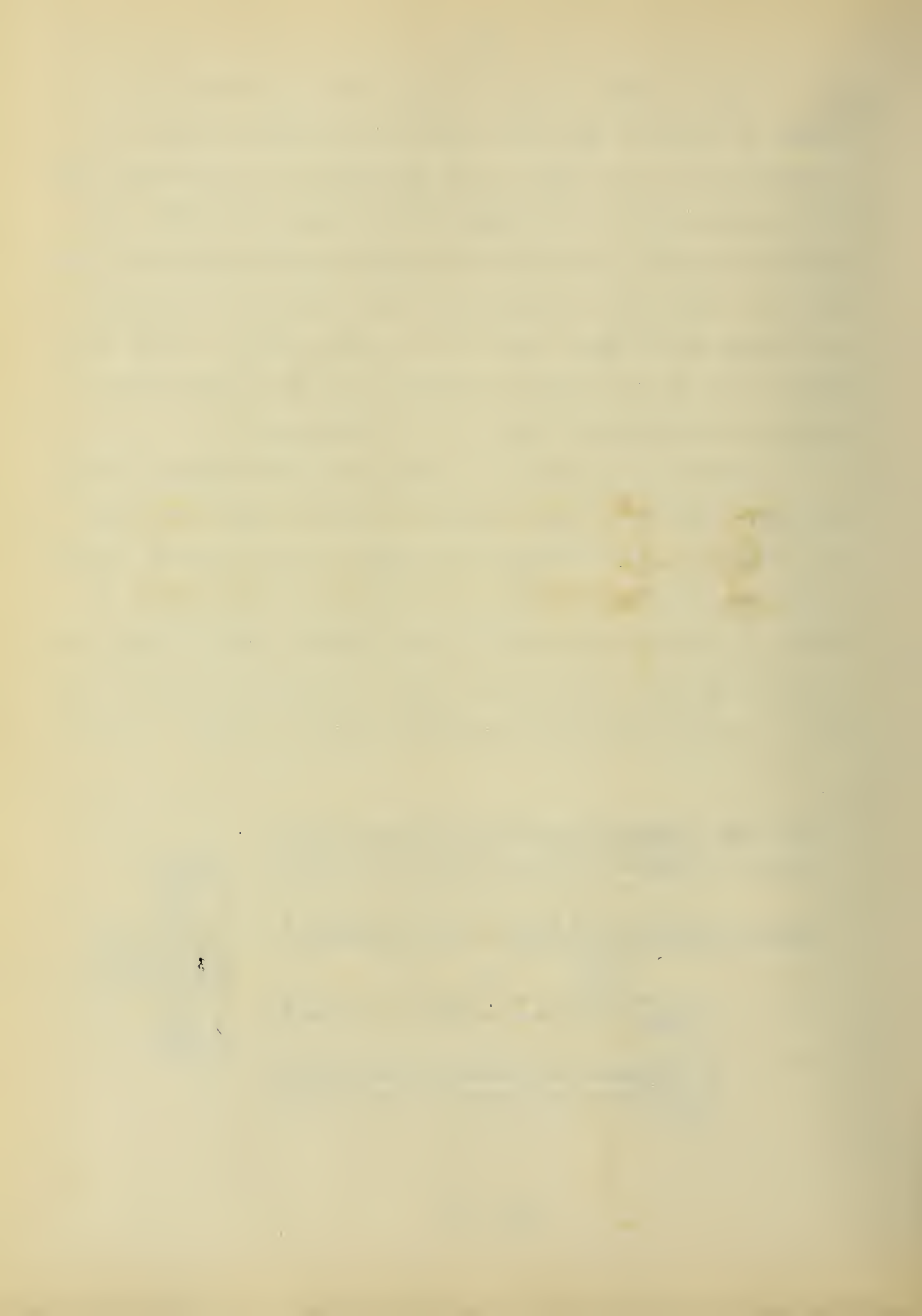


Fig. 57.



to avoid the use of sway bracing. The Plato Bridge of 273-foot span, mentioned on p. 46 has no sway bracing, each panel being 19 feet 6 inches in length.

Fig. 57 shows a form of sway bracing employed in the Mahomet Bridge, the portal of which was shown on p. 54. This represents an economical type of sway bracing, although the cutting of the channel webs adds somewhat to its expense. The form however is not rigid on account of the method of attachment of the rods. It is permissible on spans of not over 180 feet in length.

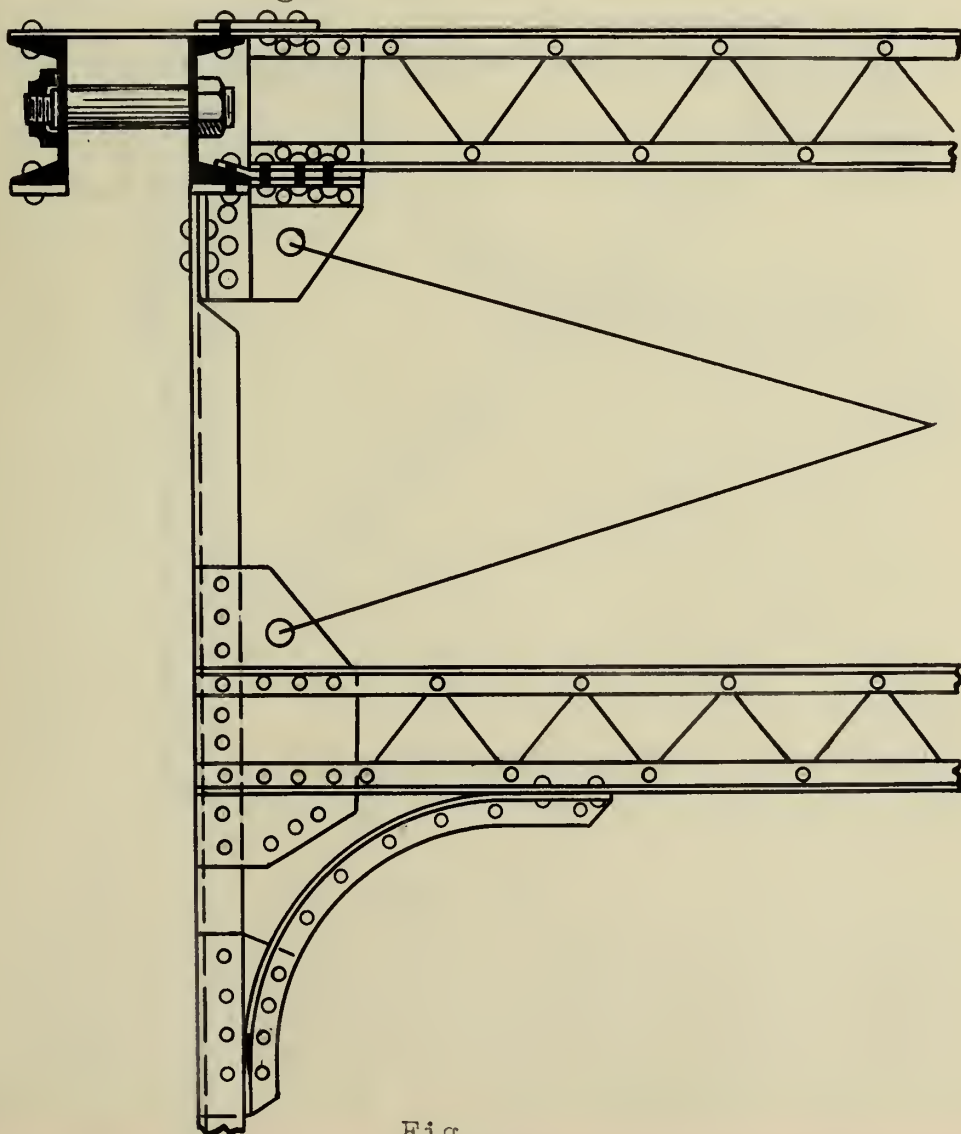


Fig. 58.



The form shown in Fig. 58 represents a better type than the preceding case. Better attachment is furnished the rods, and the placing of the strut at the lower ends of the rods adds to the rigidity of the whole. The type could be improved upon if angle bars or channels were used instead of rods, and then the structure considered as a whole would be made much stiffer. The form with the improvement suggested is, however, to be recommended and could be employed for any span.

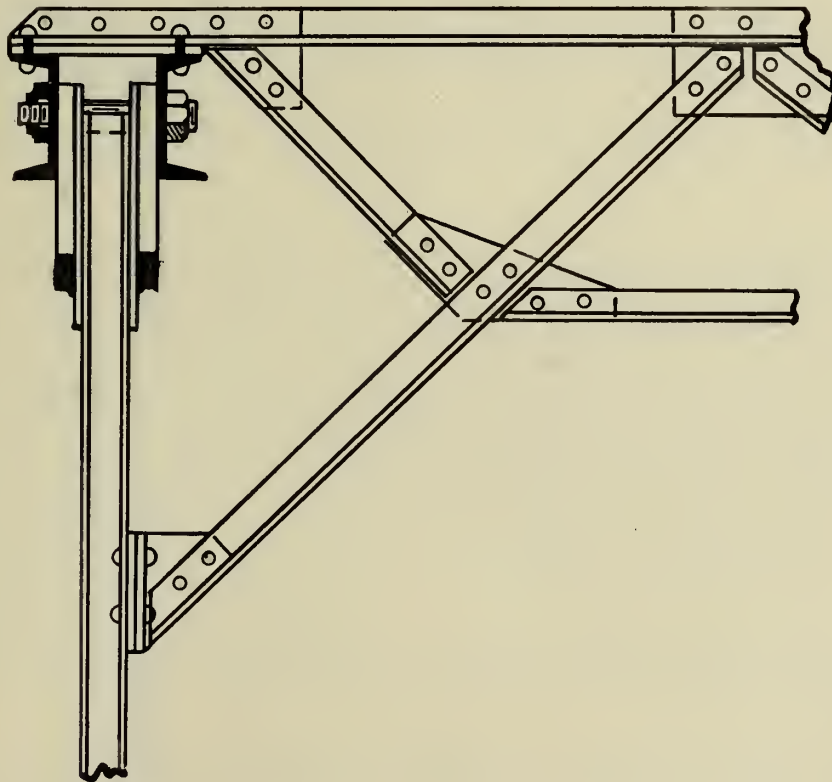


Fig. 59.

Fig. 59 also represents a very good type of sway bracing, this type being on the Maumee River Bridge, the portal of which is shown in Fig. 52 p. 50. The form could well be



adopted on all large span bridges on account of its great strength and rigidity. It is recommended for the same reasons that apply to the portal strut of the same form, and which are discussed on pages 45 and 47 .

#### ART. 11. FLOOR BEAM CONNECTIONS.

The floor beams of ordinary highway bridges usually consist of eye-beams. The eye-beams should be of sufficient depth to prevent any perceptible deflection at the middle of the maximum applied load, and should, also, have flanges of width to furnish sufficient bearing area for the ends of the joists.

All designers agree upon the eye-beam as the best for floor beam purposes; but they differ upon the method that should be employed in their connection to the other members of the bridge. As a result there are at the present date, a large number of types of floor beam connections, a great many poor ones of which are in use.

Some of the requisites of a good floor beam connection are:-

- 1st. It should distribute the stress evenly to the intermediate posts and the hip vertical.
- 2nd. It should be both strong and rigid.
- 3rd. It should be simple and economical.
- 4th. Filed riveting should be a minimum.
- 5th. It should admit of facility of erection.



6th. It should, according to Cooper's 1901 specifications, Art. 89, be so fastened to the floor beam as to prevent any end motion of the same or a tendency to rotate due to the action of the lateral system.

The floor beam connection should allow of an even distribution of stress from the floor beam to the hip vertical and intermediate posts to prevent any eccentric stresses being developed therein. If the stress is distributed evenly to them there will be no abnormal taxing of the strength of one part, for each part will take its share of its stress to be opposed. Then too, the various parts can take the stress in the direction that they were designed to take it and not in any other direction, a condition which often tends to reduce their efficiency, as would be the case where eccentric stresses would be developed. Eccentric stresses often cause rivets to take tension. This is not allowable according to Cooper's 1901 specifications.

A floor beam connection should also be both strong and rigid. It should be amply strong to resist all of the stress that may be brought to bear upon it. It should be rigid enough to transfer to the end post all of the stress that is developed in the floor beam. It should be simple in order that it be economical; greater stress can also be transferred through a simple connection than through a more complex one. A good connection should be economical in that it should not be abnormally expensive as compared with the other parts.

Field riveting should be reduced to a minimum in order



that the form be economical. An excessive amount of field riveting indicates one of two things, either that the connection is weak, or it has an unnecessarily large amount of material in order to give sufficient riveting space for the extra number of rivets that are required for the field riveting. Cooper in his 1901 specifications considers field rivets only two thirds as effective as shop rivets, since the allowable stress that he gives for field rivets is but two thirds of that for shop rivets. Therefore, if a certain number of rivets are to be driven in the field instead of in the shop, it would require half again as many rivets, and this in turn would mean a great deal of added material to furnish the added riveting space which would be necessary in that case.

The floor beam connection should admit of facility of erection in order to reduce the cost, and thereby be the more economical.

It should effectually stay the floor beam from any end motion or any tendency to rotate from the reaction of the lateral systems. If, for instance, a small amount of rotation were allowed the floor beam might be thrown off of center and eccentric stresses be thereby developed with the same results mentioned in the first paragraph of this discussion.

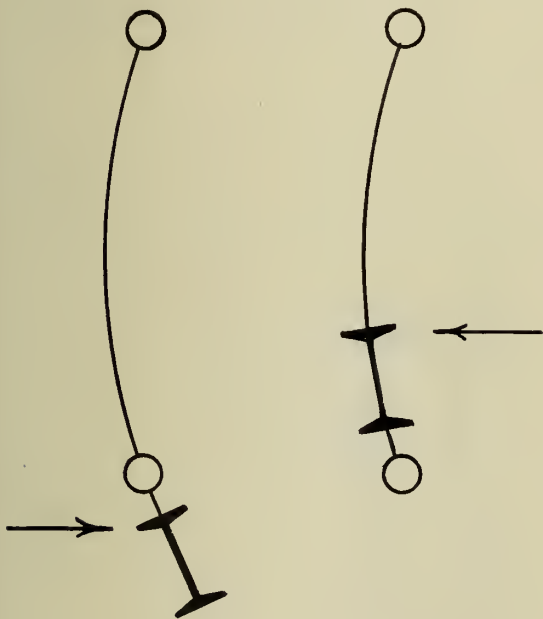
There are two distinct types of floor beam connections in use at present; those that are used with posts having the channel webs perpendicular to the roadway and those that are used with posts having channel webs parallel to the roadway. For bridges having posts of the former type, the better con-



nection is offered.

The use of floor beam hangers is not to be encouraged on account of their inefficient mode of connection and the lack of rigidity. They should only be used on the smallest spans. Cooper states in his 1901 specification that all floor beam hangers, when permitted shall be made without adjustment, and so placed that they should readily be examined at all times.

Floor beam connections may be employed either above or below the chord pins. Fig. 60 shows very plainly what occurs



when a passing load stops upon a bridge where the floor beams are located below the chord pins, while Fig. 61 shows the result for the same case when the floor beams are located above. In the latter case the floor beam tilts in the same direction as the post, and serves as a great stiffener for that member. Then too, the transmission of stress from the floor beam to the post is better on account of the shorter distance that it has to travel. Neither

Fig. 60.      Fig. 61.      does the latter case require as long a channel as the former and is, therefore, more economical, as the increased material required in the portal when the knee brace is placed higher up to give sufficient headroom, will not balance the amount of



material saved by the placing of the floor beams above the chord pin. The use of the floor beams as shown in Fig. 61 also tend to make the posts much stiffer in the resisting of wind strains. Therefore, since the method of placing floor beams above the chord pins is both more economical and efficient than the placing of them below, it is to be recommended in all cases.

A comparison of the various types of floor beam connections follows, those in which the floor beam is below the chord pin being taken up first.

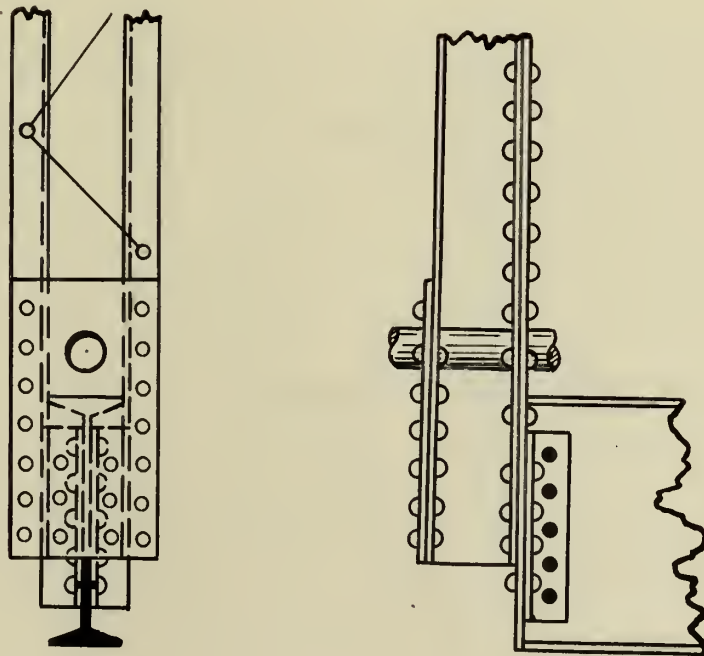
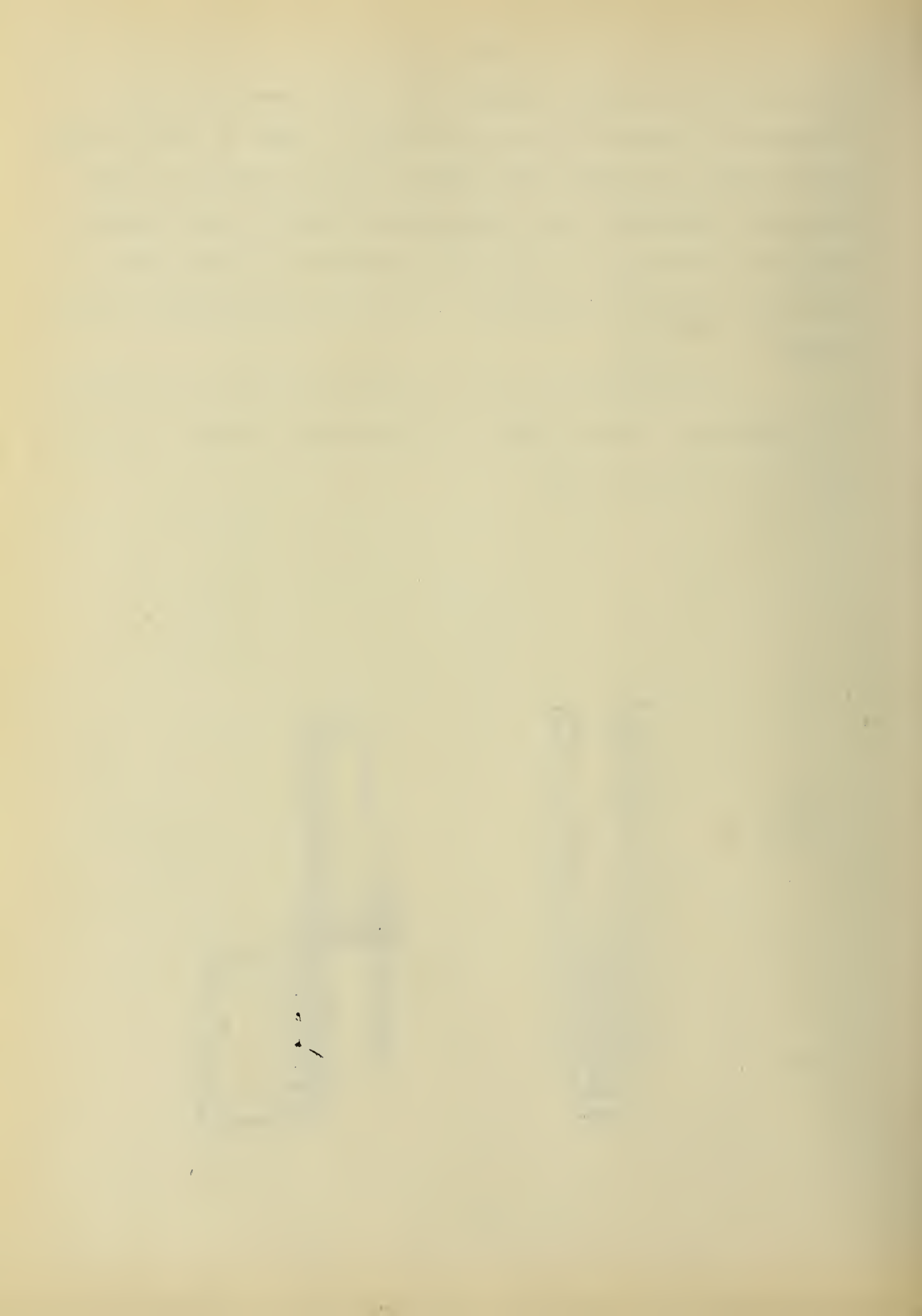


Fig. 62.



The connection shown in Fig. 62 represents a fairly common type of floor beam connection, ten cases being found. It can only be used where intermediate posts have their channel webs perpendicular to the roadway. The stress is not transmitted evenly from the floor beam to the pin, since the former has only one pin plate through which it can directly transfer stress. The form, however, can be made very rigid by the extension of the channels farther down, thereby offering greater area for rivets. This will cause a more uniform distribution of stresses through the pin plates. Although the above method of obtaining a rigid connection increases the cost of the connection, it is, nevertheless, the best connection in present use where floor beams are located below the chord pins, and intermediate posts have their channel webs perpendicular to the roadway.

Fig. 63 shows a form of connection to be found where intermediate posts have their channel webs parallel to the roadway. It is found in bridges of long span and of large panel lengths. It represents a successful but expensive attempt to use a connection which does not properly belong to bridges having posts as shown in the accompanying figure. It is virtually the same connection as that shown in Fig. 62, but applied to the case where channel webs are parallel to the roadway. It requires heavy pin plates, the avoidance of the use of which is one of the reasons that channels in posts are placed with their webs parallel to the roadway. Rather than have the added cost of a second pair of channels for the floor



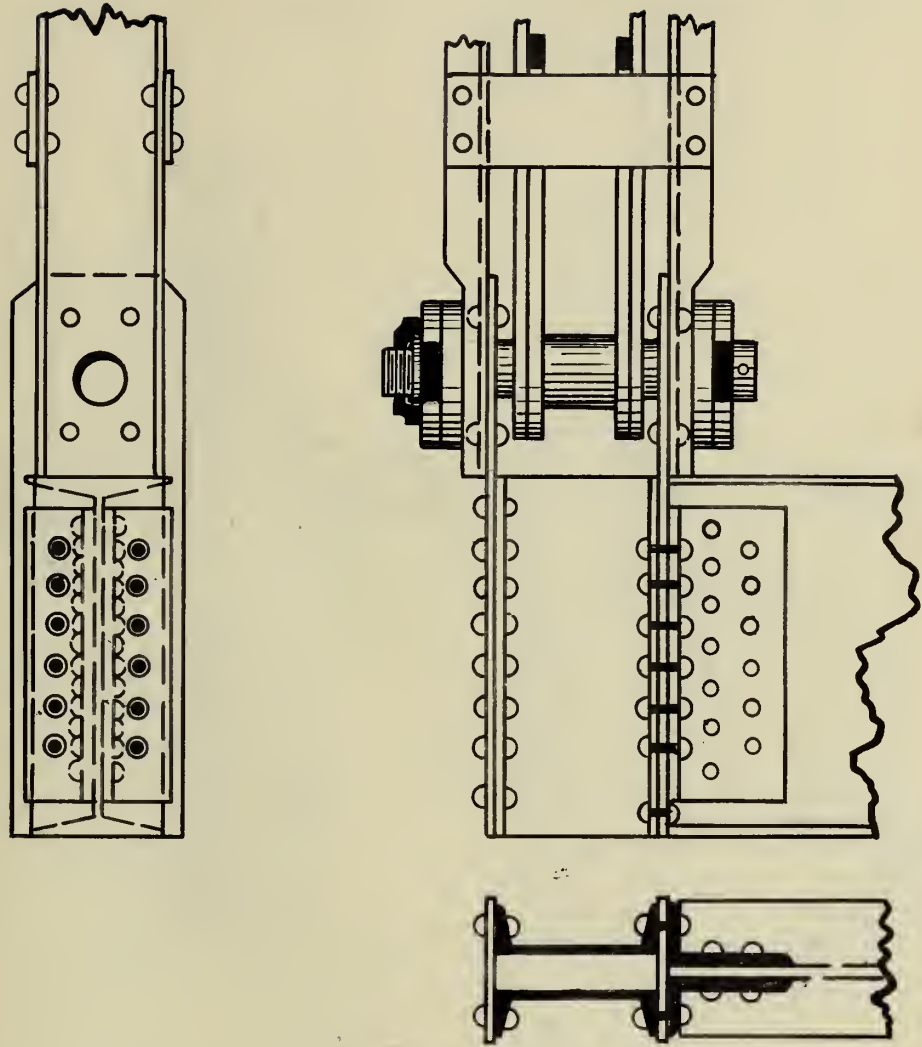


Fig. 63.

beam connection, why not place the channels with their webs perpendicular to the roadway in the first place. It is an efficient connection, but too expensive and is, therefore, not to be recommended.

A very common form of floor beam connection and one similar in principle to the proceeding connections is shown in

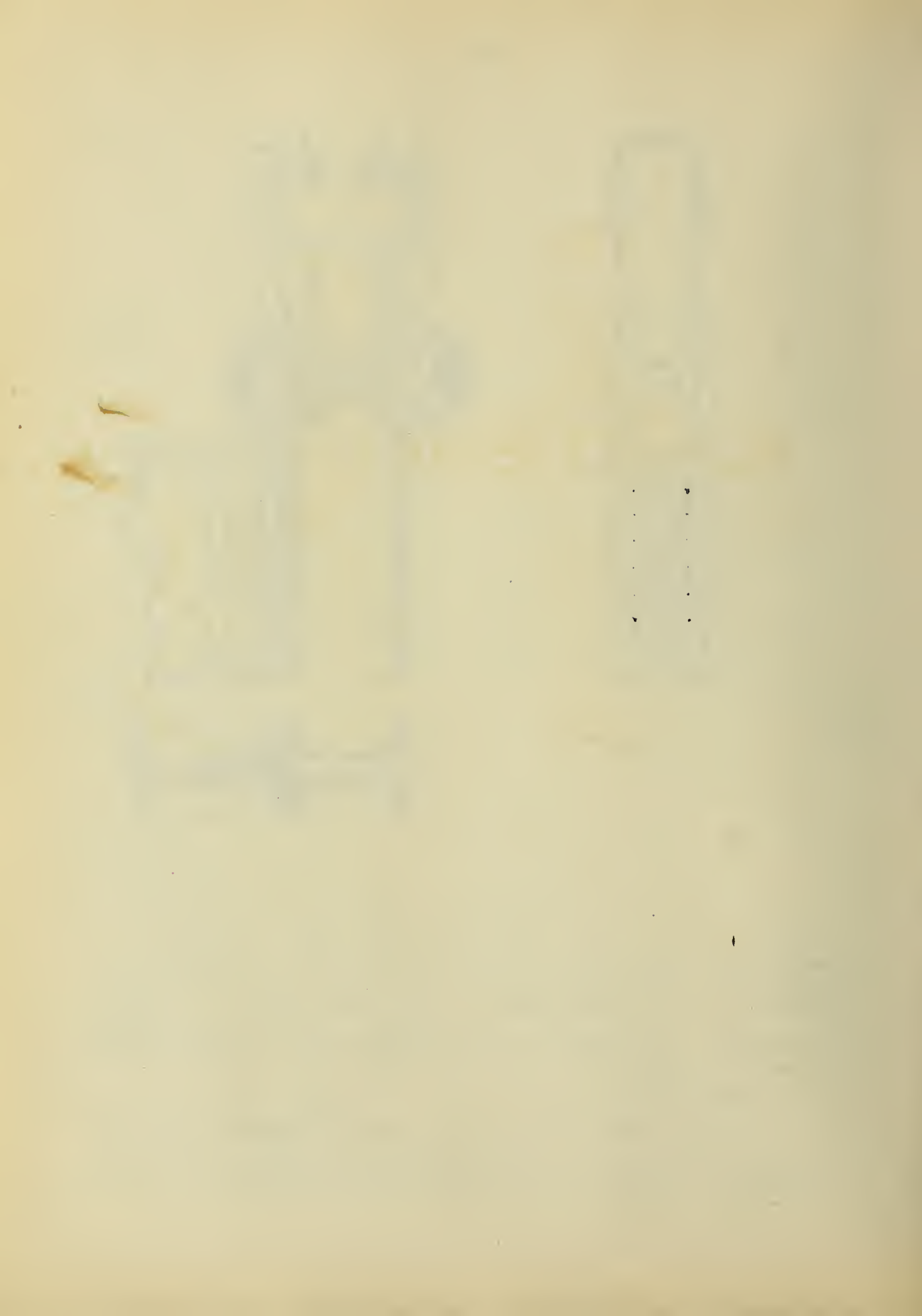


Fig. 64. This form of connection is efficient, and sufficiently rigid to transfer the comparatively small stresses from the

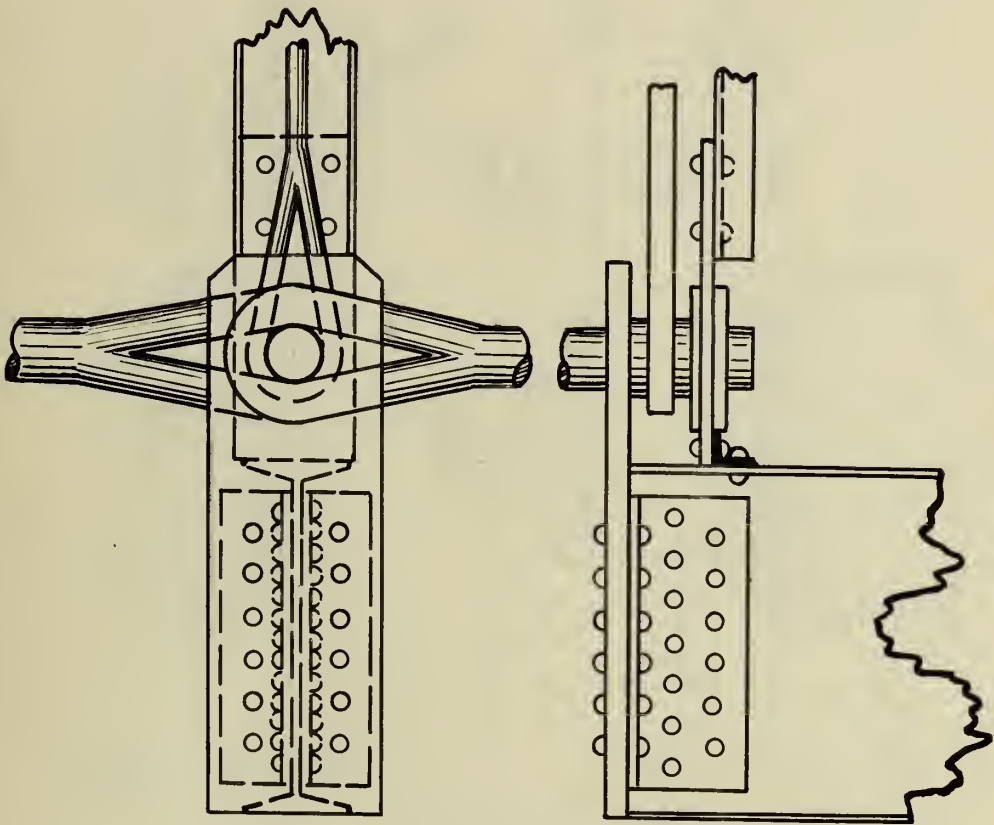


Fig. 64.

floor beam to the hip vertical. The one great fault with this connection is that it usually causes a very large bending moment on the pin, as the hanger is almost always located at the center of the pin. The added expense of the large pin required is however quite small. Then too, the form possesses but little rigidity and is for the great part but a modification of the ordinary type of floor beam hanger. The detail is, therefore, not to be recommended for reasons explained on p. 61.



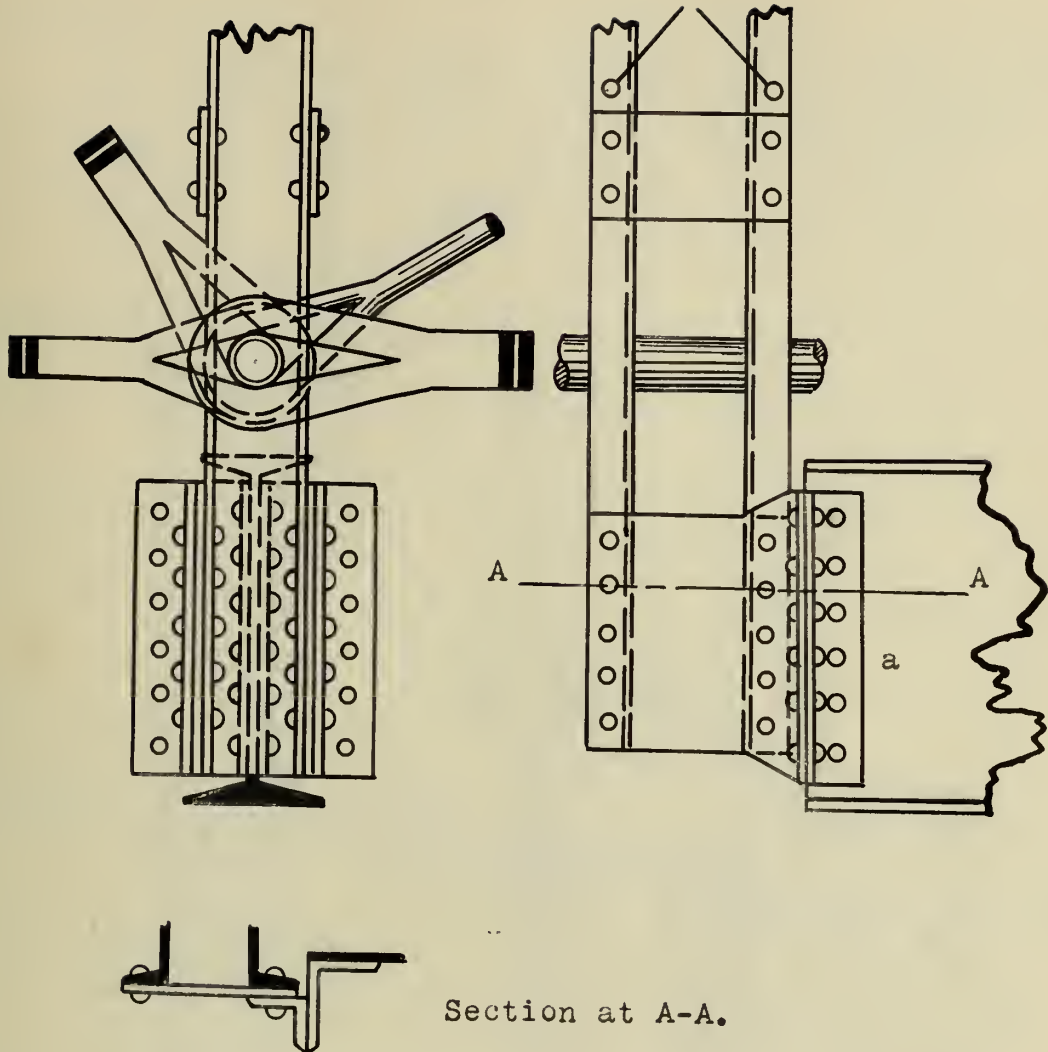


Fig. 65.

A somewhat rare form of connection is shown in Fig. 65, only four cases of this type being found. It was used on some of the largest bridges, the chief one being the 273-foot Plato Bridge, mentioned on p. 47. It cannot be said to be an expensive form, since it is composed entirely of angles and plates, although their number is quite large, half a dozen pieces being required. The connection is quite rigid although there are five connections through which the stress must be



transferred.

The connection would be improved if the angle at "a" had a double row of rivets. The form is to be recommended.

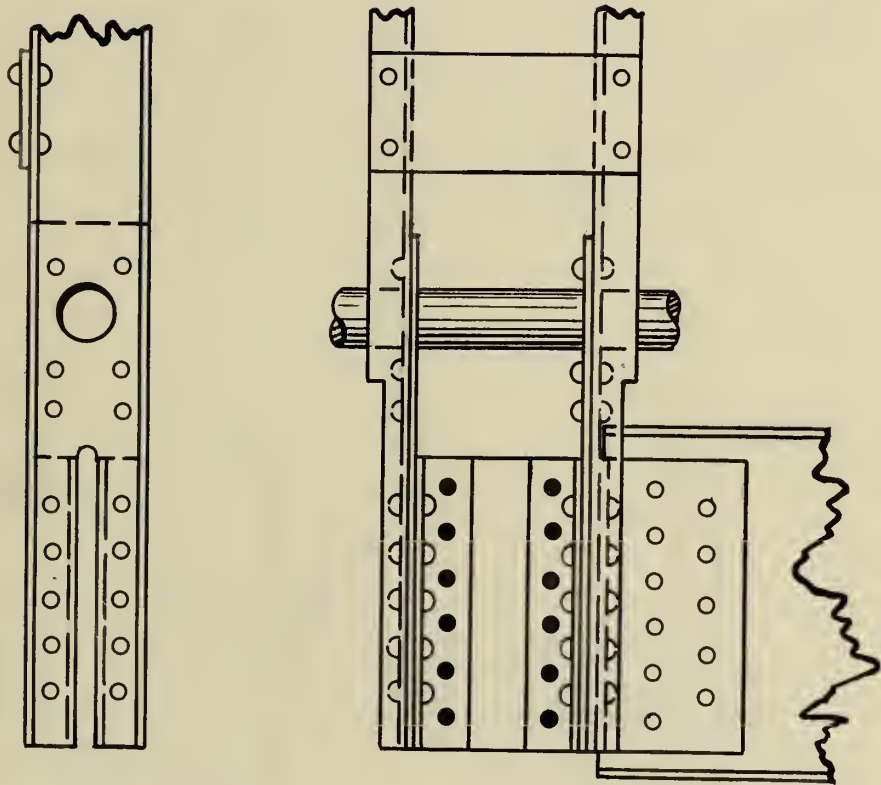
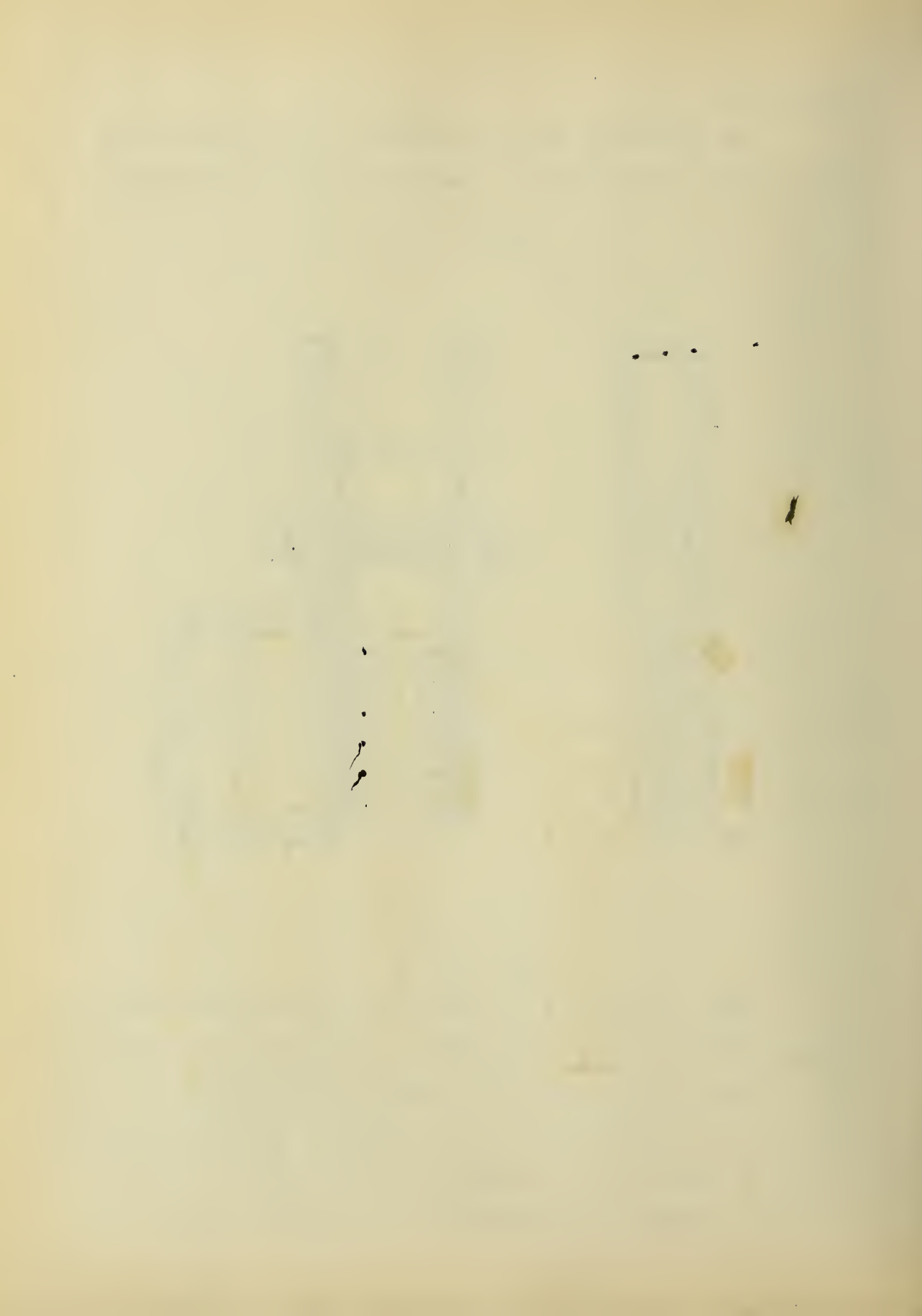


Fig. 66.

Fig. 66 shows a very effective, although expensive form of connection found in eight cases. Its great cost rises from the cutting of the slots in the channel webs and from the cutting down of the eye-beam. This connection approaches the ideal as far as possible for a connection of this type since it admits of a fairly even distribution of stresses from the floor



beam to the pin on account of the floor beam being in direct connection with both of the channels. An exactly even distribution of stress from the floor beam to the pin cannot be obtained, since for any deflection of floor beam the greater stress will be transferred through the nearest channel. The connection angles shown in the figure make the form very rigid, and they are very capable of transferring stress on account of the large riveting area that is offered. The type is to be recommended for large spans on bridges having posts with their channel webs parallel to the roadway.

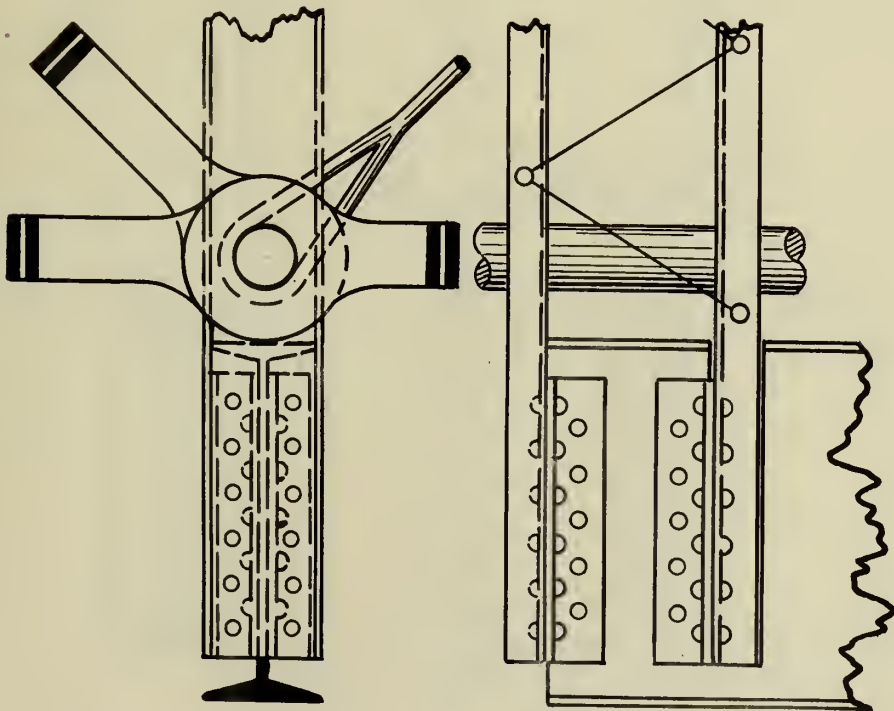


Fig. 67.



Both a poor and at the same time an expensive connection is shown in Fig. 67. It is expensive on account of the cost required to cut the eye-beam. The cutting of the eye-beam must weaken the connection considerably and make what is gained in even distribution of stress be lost in the general weakness of the connection as a whole.

This connection could not be safe on any bridge employing panels of over 25 feet in length, and is not to be recommended for reasons above stated.

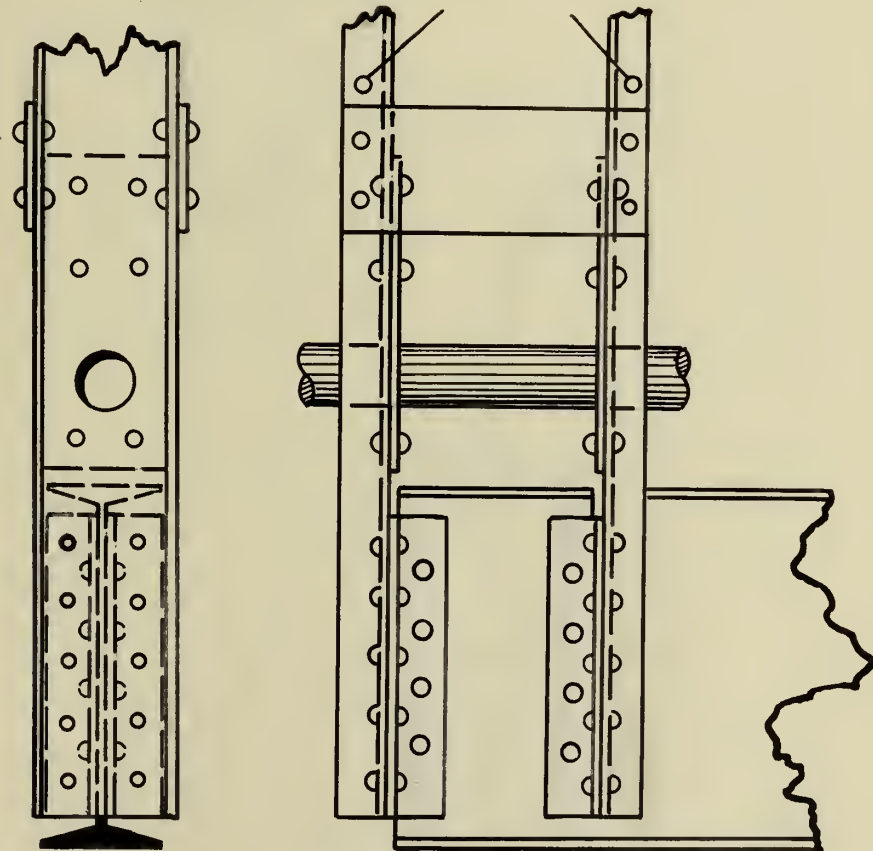


Fig. 68.

The form shown in Fig. 68 represents a much better con-



nection. This form might be said to be somewhat expensive, but not more so than that shown in Fig. 66 where much more material is required. It is stronger than the detail shown in Fig. 67 since the floor beam is not cut down any in its depth, this being the place where most of its strength lies. Its method of connection to the channels is just as efficient and therefore the form is to be recommended for any span where floor beams are placed below the chord pins.

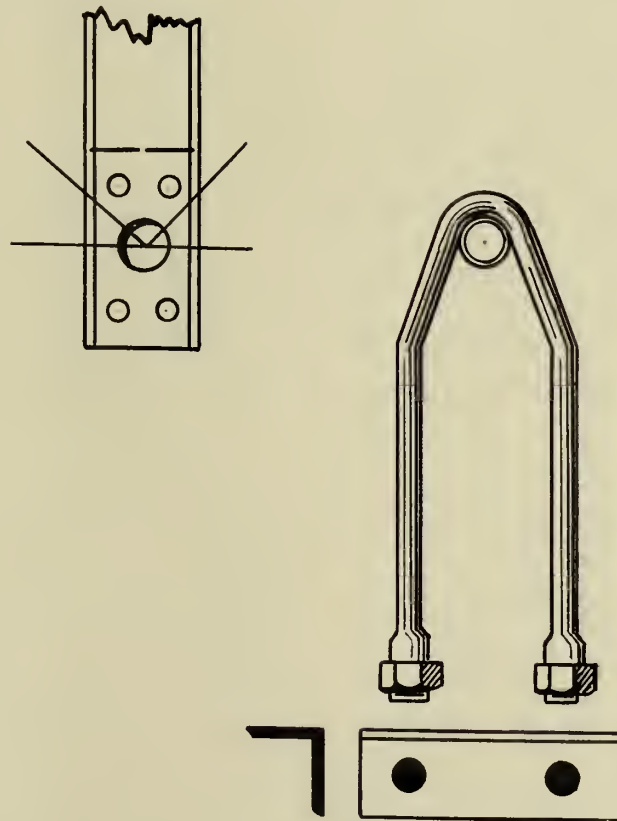


Fig. 69.

A very common form of connection is shown in Fig. 69, this being found in sixteen cases. Its use is only permissible for the smallest spans, on account of its lack of rigidity, and



the large bending moment that it creates on the pin. The floor beam resting on "a" is allowed to slide backward and forward in a direction perpendicular to the plane of trusses, since there is not a fastening at either end to prevent such a motion. However this movement is prevented to a small degree by the weight of the floor system, and the connection at equal intervals of the joists to the floor beams. A large bending moment is usually created in the pin, for the hanger is generally placed at its middle, thereby necessitating the use of a pin of abnormal diameter. Then too, a circular rod allows but little area of contact which causes the hanger to wear away fast, thereby endangering the floor system. This fault could be remedied by using a rectangular bar instead. Also, a hanger admits of a certain amount of swinging of the floor beam when a live load passes over the bridge. Therefore, it is the opinion of the writer that a hanger of this type should never be used.

Fig. 70 shows an improved form of the same type which meets the objection mentioned on the proceeding page, that the floor beam might slide backward and forward. It is to be noticed that a plate instead of an angle as in Fig. 69, is used. An angle should be used since by its depth, it prevents any deflection which might occur where a plate is used, thus reducing the total amount of vibration. Notwithstanding the above mentioned improvement the form is not to be recommended for other reasons which are mentioned on the proceeding page under the discussion of Fig. 69.



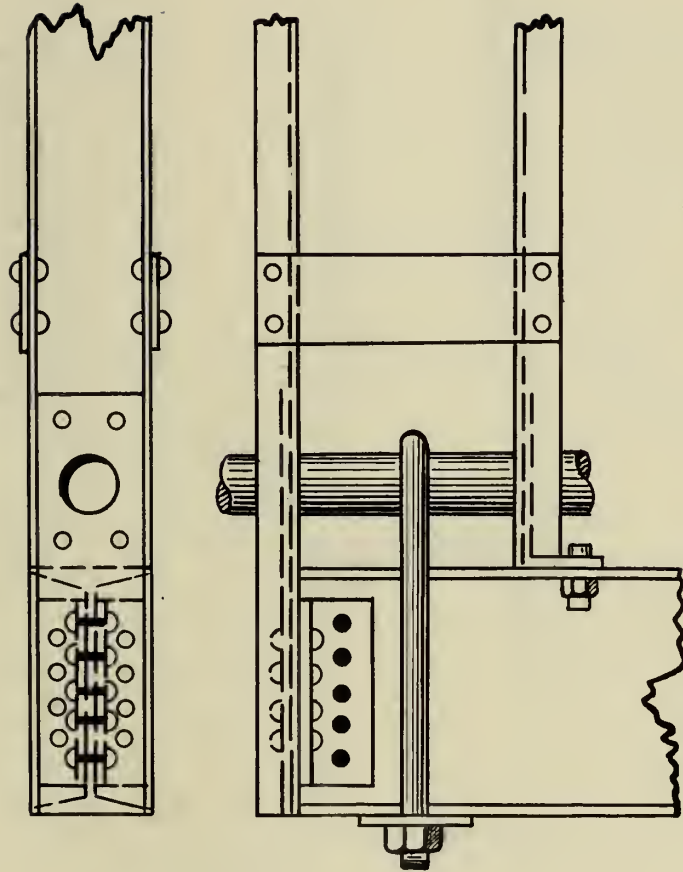


Fig. 70.

For bridges having the floor beams located above the chord pins, an economical and efficient connection is offered in the form shown in Fig. 71. The form is economical since only two angles are used. It is efficient on account of the large number of rivets which may be used to fasten the leg of the angle to the flanges of the channels. The pin plates



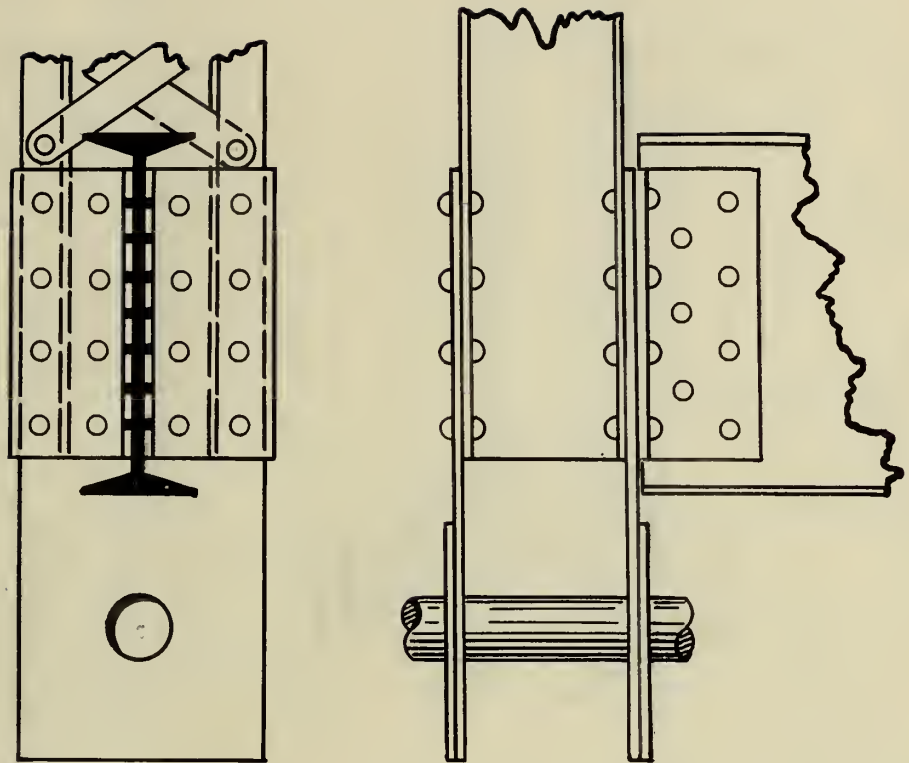


Fig. 71.

make the section of the post very rigid at this part, which rigidity in turn means the more even distribution of stress from the floor beam to the pin. This connection offers a very even distribution of stresses, and is to be recommended.

Fig. 72 shows a form of connection very similar to that shown in Fig. 71, the only difference being that the channels instead of being cut off at a point even with the lower part of the floor beam are extended below the chord pin.



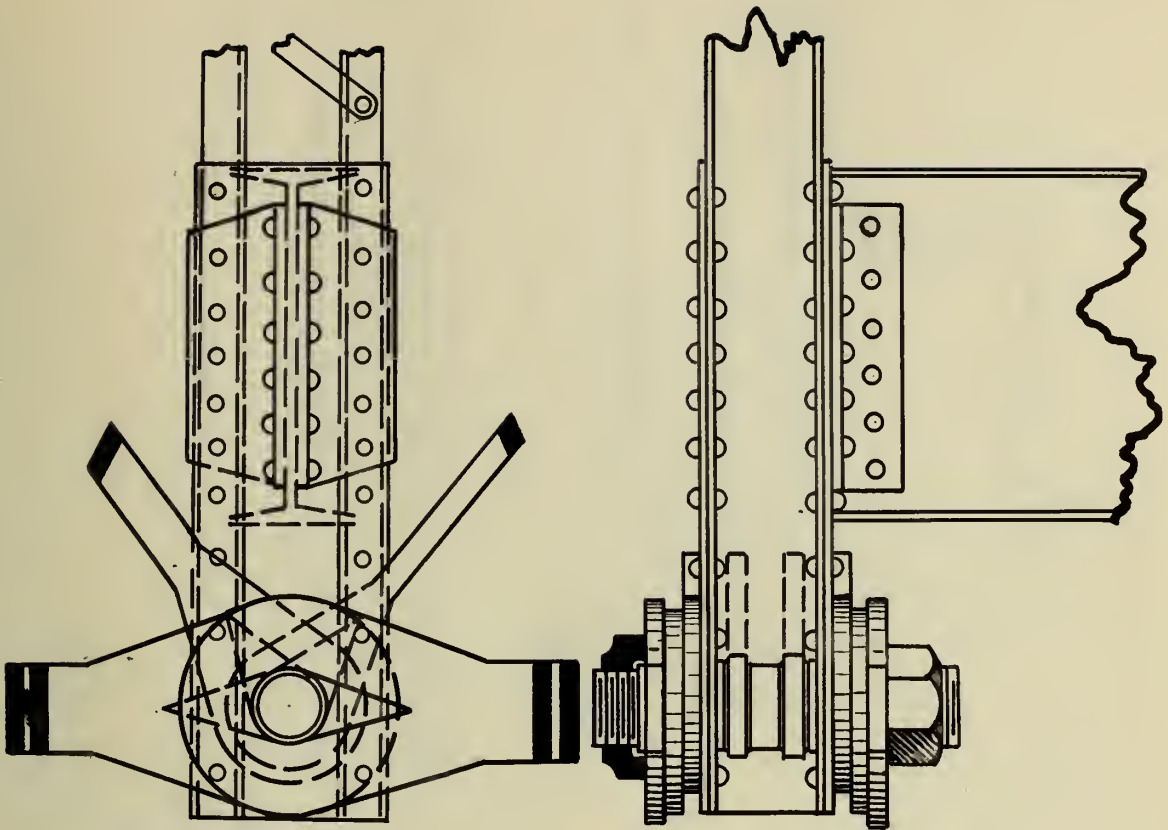


Fig. 72.

This offers a greater amount of area for riveting the pin plates to the flanges of the channel, and might be said to offer a little more even distribution of stresses, both on account of the increased rigidity offered, and the fact that some of the connecting rivets might be driven closer to the pin, than in the form shown in Fig. 71. This form should only be employed for the longer panels - panels of over 25 feet in length.



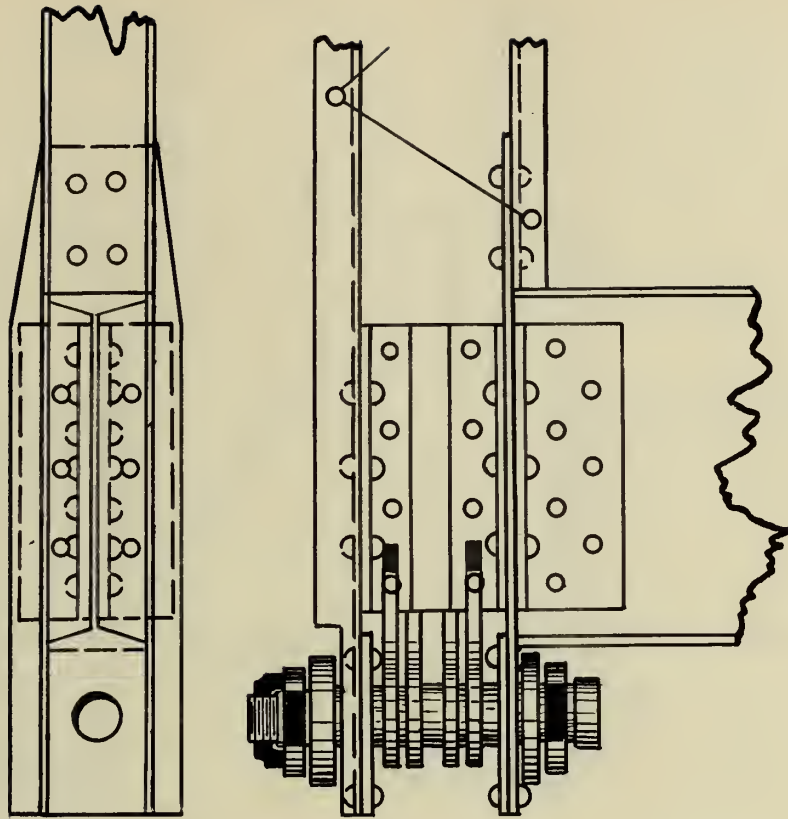


Fig. 73.

A less efficient but more common form of connection is shown in Fig. 73. The form is more expensive than that shown in Fig. 74 since more material is required, six angles and one plate being used.

Fig. 74 shows a somewhat rare but economical and efficient form of connection. This detail is economical in that it requires but two plates and two angles. The two plates which connect the flanges of the channels give it a large radius of gyration and, therefore, great stiffness about an axis perpendicular to the roadway. This is required to resist the deflections of the floor beam. The connection



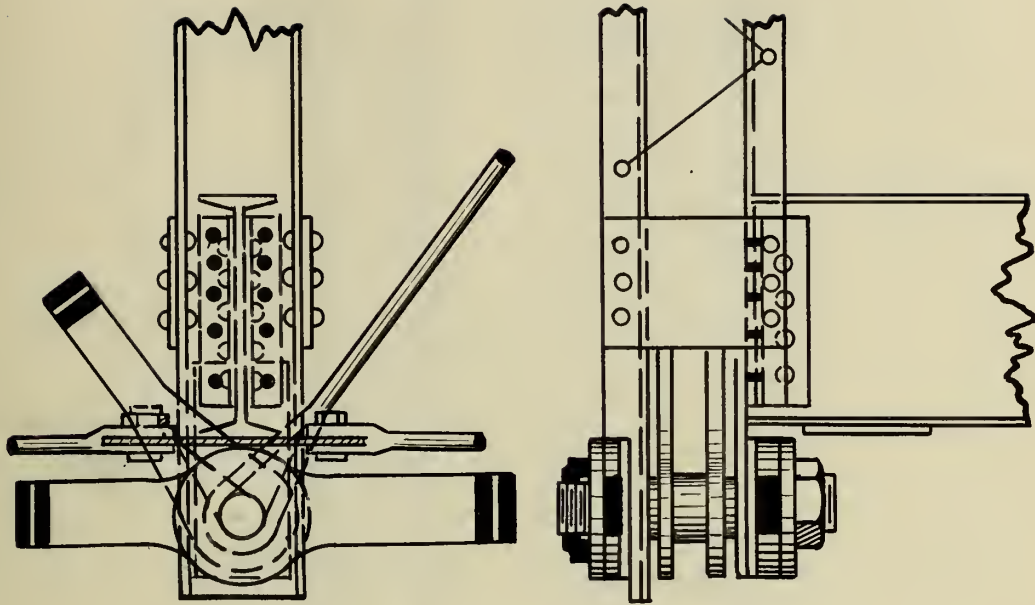
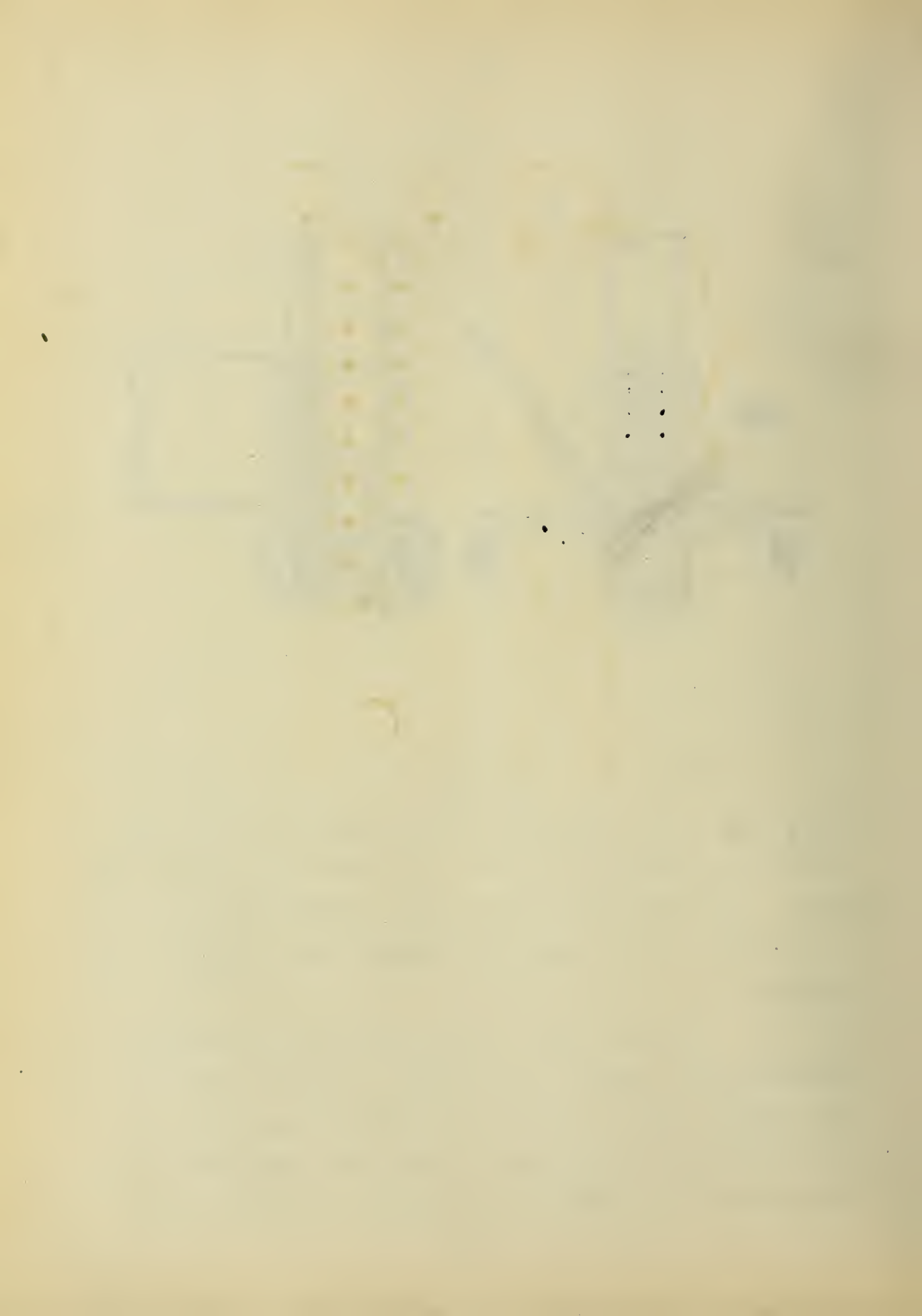


Fig. 74.

being sufficiently rigid about both axes, a very even distribution of stresses from the floor beam to the post is obtained. The form is, therefore, to be recommended for bridges of all spans, having posts with channel webs parallel to the roadway.

Fig. 75 shows a very good form of connection employed where channel webs of posts are perpendicular to the roadway, the floor beam in this case being made up of four angles and a web plate. A very rigid connection of the floor beam to the pin plate is offered on account of the



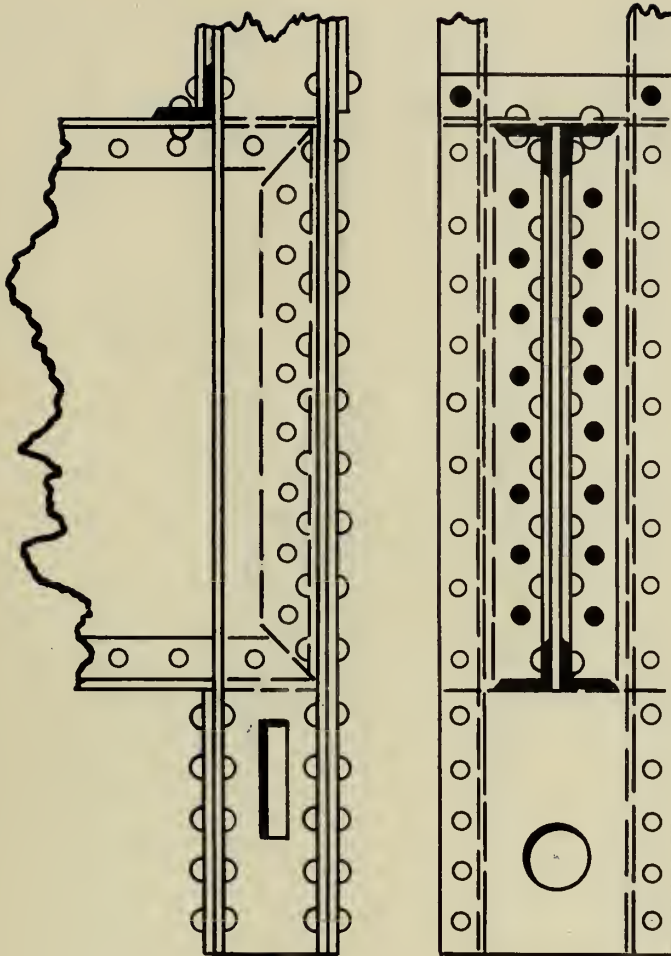
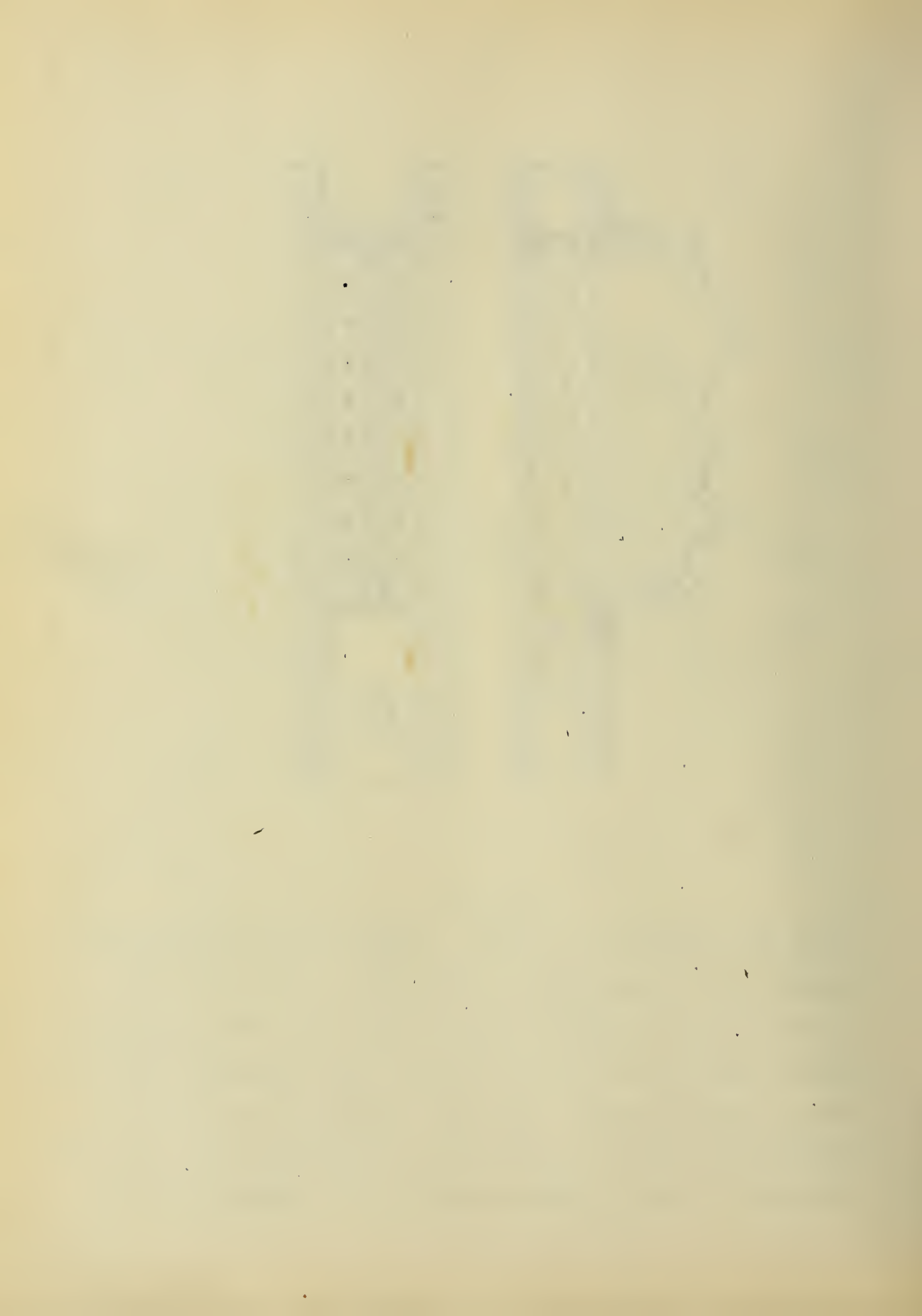


Fig. 75.

large area allowed for riveting. There are two pin plates to transfer the stress from the floor beams to the pins. The stress is more evenly distributed than in the form shown in Figs. 71 and 72, since both pin plates take compression when a load is applied on the floor beam; whereas in the above mentioned forms the inner plates take compression, while the outer ones have a tensile effect produced by the deflection of the



floor beam which thus prevents an even distribution of stress. The form is also economical in that only two angles are required in addition to the necessary pin plates. The form is indeed to be recommended on account of its being both strong and economical, and is recommended for bridges of all spans.

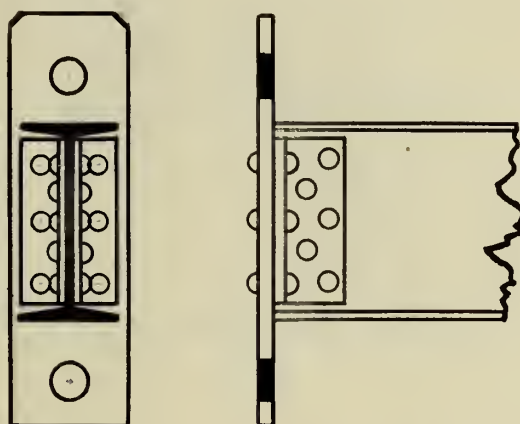


Fig. 76.

A common form of connection of the floor beam to the hip vertical is shown in Fig. 76. It is at once noticed that two pins are used, the upper one connecting the bars of the hip vertical to the hip of the upper chord, while the lower one is used for the lower chord connection. It is built on much the same principle as the hanger and therefore for reasons stated on p. 61 should not be used. It should not be used even for hip vertical purposes, for bridges of over 100-foot spans on account of its lack of rigidity, which arises



from the absence of any method to prevent the floor beam from slipping back and forth in a direction perpendicular to the roadway.

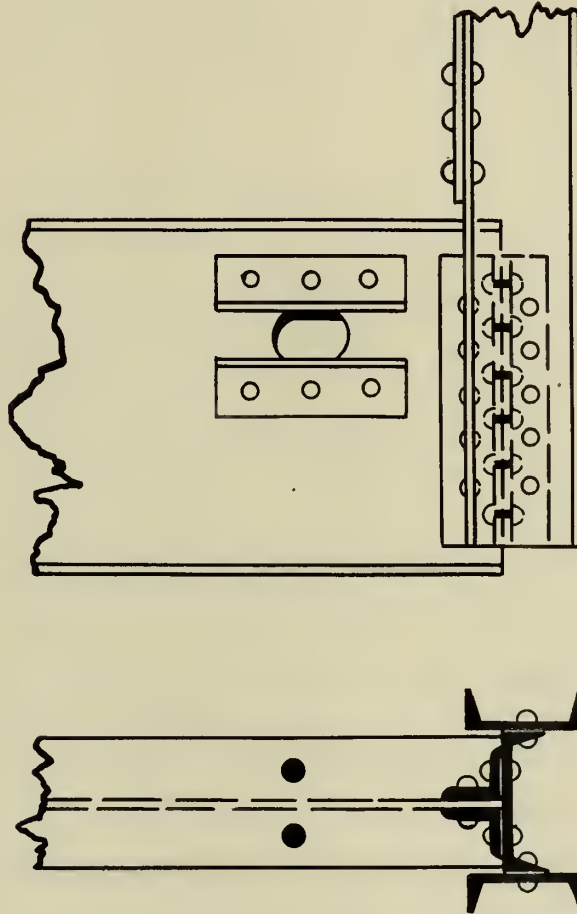


Fig. 77.

A rare but very efficient form of floor beam connection is shown in Fig. 77. This form being found employed on a bridge of 105-foot span built in St. Joe Twp., Champaign Co., by the Lafayette Bridge Co. The detail offers a rigid connection of floor beam to post and also allows a uniform distribution



of stress to same. It is not expensive since only one channel and two angles are employed. The amount of rigidity developed in the connection depends upon the depth of the floor beam, since the deeper the floor beam the more riveting area is offered. The form is recommended for all spans.

#### ART. 12. JOISTS.

Joists are used for the support of the floor system. When wooden bridges were in vogue they merely consisted of planks which were laid end to end in much the same manner that the joists of the present time are. At the present time joists are made up of either eye-beams or channels or both. These are laid in equally distant rows parallel to the roadway, the ends of each resting upon the floor beams. Eye-beams are usually used for all the inside rows and channels for the two outer ones. The floor-beam flanges should be of sufficient width to afford ample footing for the joists, and also allow at least one half of an inch distance between ends of joists to allow for changes of temperature and inequalities of length. If this distance is not left between the joists, a high temperature might cause the eye beams and channels to expand and meet each other, and then buckling will result. Rivets or bolts are used to fasten the joists to the floor beams and these should never be placed less than one and one-fourth inches from the ends of the joists. In designing joists, they should be considered as simple beams acting under a



uniform load. A wooden plank is usually fastened to the top of each joist by means of bolts which are placed at equal intervals along the joist's length, as shown in Fig. 78. This is to give nailing area for the fastening of the floor to the joist system.

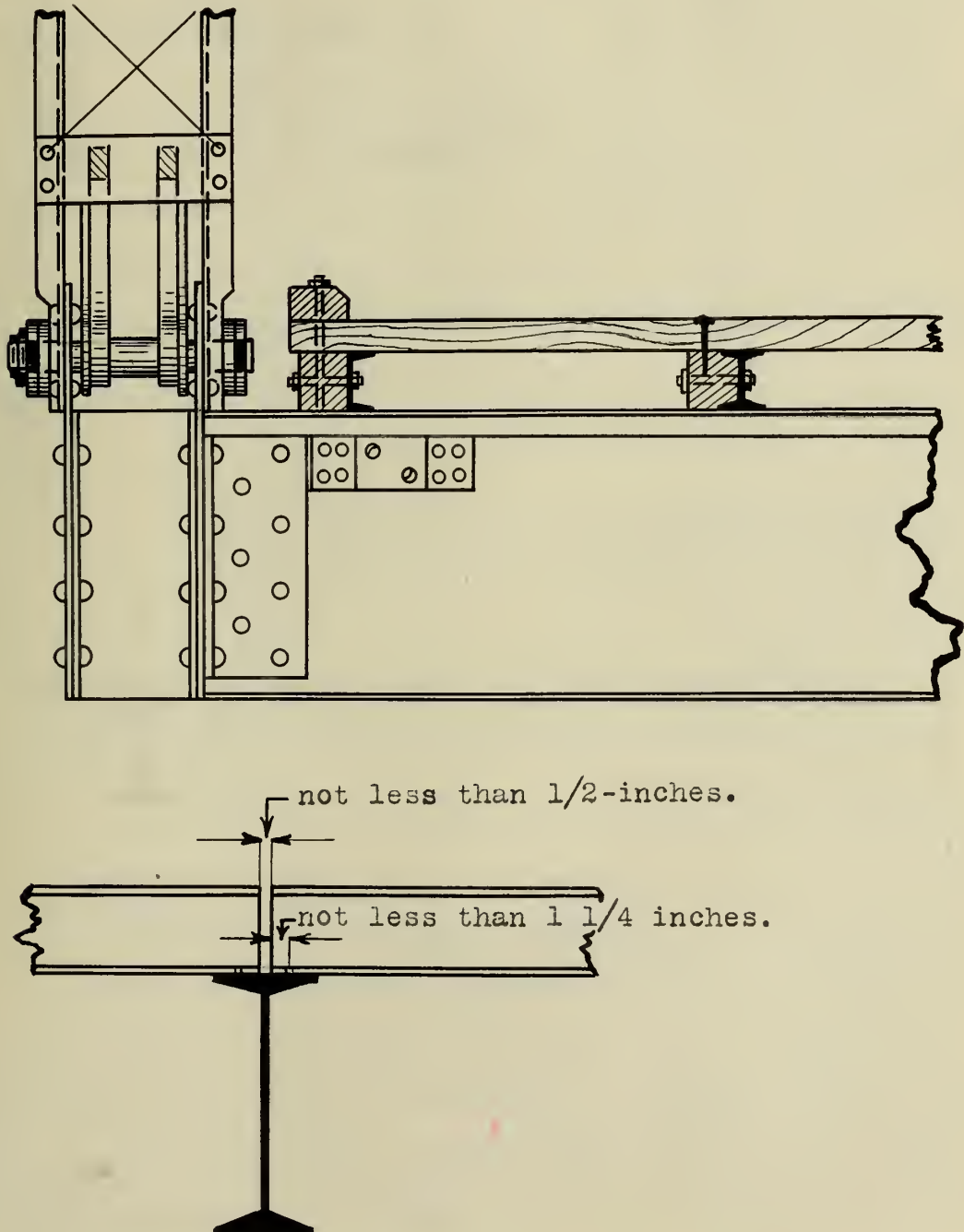


Fig. 78.



ART. 13. JOIST RAISERS and SHOE STRUTS.

Joist Raisers are employed at the end of highway bridges to keep the joists and flooring in the end panels level. The shoe strut is used to support the joist raisers which in turn support the flooring. Very often the joist raisers are dispensed with, in which case the shoe strut either consists of an eye-beam or a channel. In the majority of cases however, joist raisers are used. They usually consist of channels which are placed upon the shoe strut which is generally made up of either channels or eye-beams. The most common types of end struts follow.

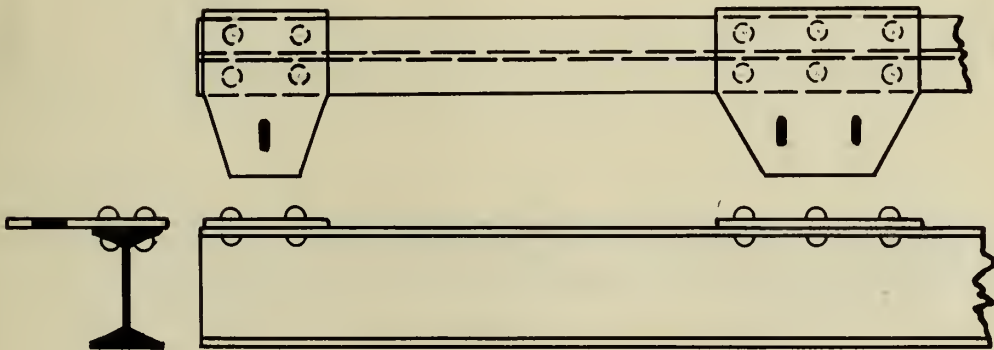


Fig. 79.

A very common and efficient form of end strut is shown in Fig. 79 . It is economical in that only one piece is necessary, this being an eye-beam. A modified form is found where a number of plates are fastened to the upper flange to give attachment to the joists. This is shown by the dotted lines in the above figure. The form offers a good connection



to the pedestal by the upper flange of the eye-beam. For long panel lengths, it does not give sufficient bearing upon the masonry, since the shoe strut has to support one half of the panel load. Channel forms, such as those shown in Fig. 80 should be used instead. Both forms allow for expansion at the roller end. The first form allowing for it by the slot which is cut in the connecting plate, the second by the slots in each joist connecting to the plates. The form is to be recommended for bridges employing panel lengths of not over 15 feet.

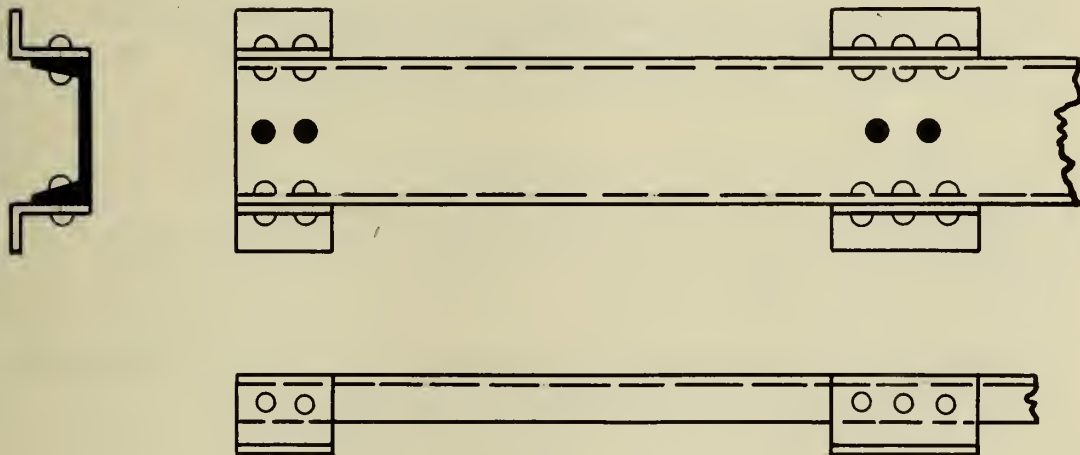


Fig. 80.

Fig. 80 shows a somewhat common form of combined strut and joist raiser it being in this case a channel. These angles are riveted to the flanges of the channel and serve to give the strut rigid and even bearing on the masonry. The detail is quite rigid and offers a good connection to the pedestal by means of the web of the channel, thereby increas-



ing the rigidity of the entire bridge structure against the wind. It affords not only sufficient opportunity for a uniform distribution of stress from the floor system to the joist raiser, but also a good distribution of stress to the masonry, since the area of contact between the angles and the masonry is ample. The form is efficient for bridges of short panel lengths where a great stress is never obtained in the shoe strut.



Fig. 81.

The form of end strut shown in Fig. 81 is more common than that shown in Fig. 80. A plate rests upon the masonry throughout the entire length of the strut. This plate supports a channel on its flange ends and thus transmits the stress from the channel to the masonry plate. The plate is used to distribute the stress uniformly to the masonry. The form would be more efficient if the channel were held rigidly in its place, but as it is there is nothing but the weight of the flooring to prevent its moving too and fro parallel to the roadway. This form can therefore be used for the roller end as well as for the fixed end. The form is not to be recommended as very desirable.



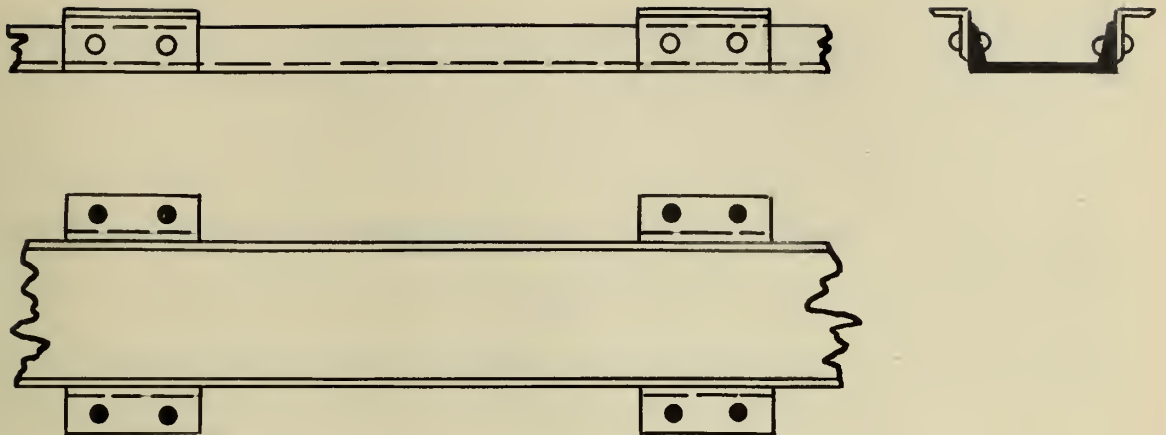


Fig. 82.

A rare though efficient form of end strut is shown in 82. This is reverse of the strut shown in Fig. 80. In this case the shoe strut consists of the channel, and the joist raisers of angles which are riveted to the flanges of the channel. The stress from the floor system is distributed quite evenly to the strut, which in turn distributes it uniformly to the masonry. The form is rigid, is not expensive and is to be recommended.

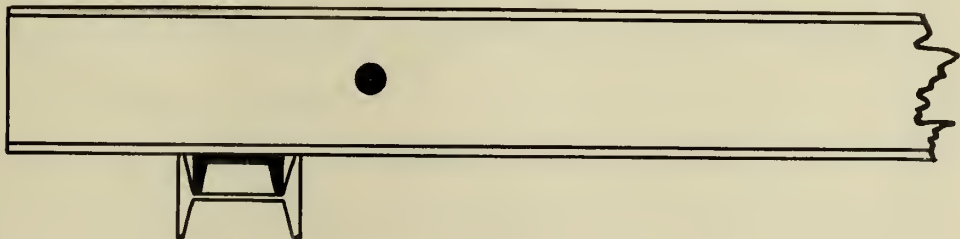
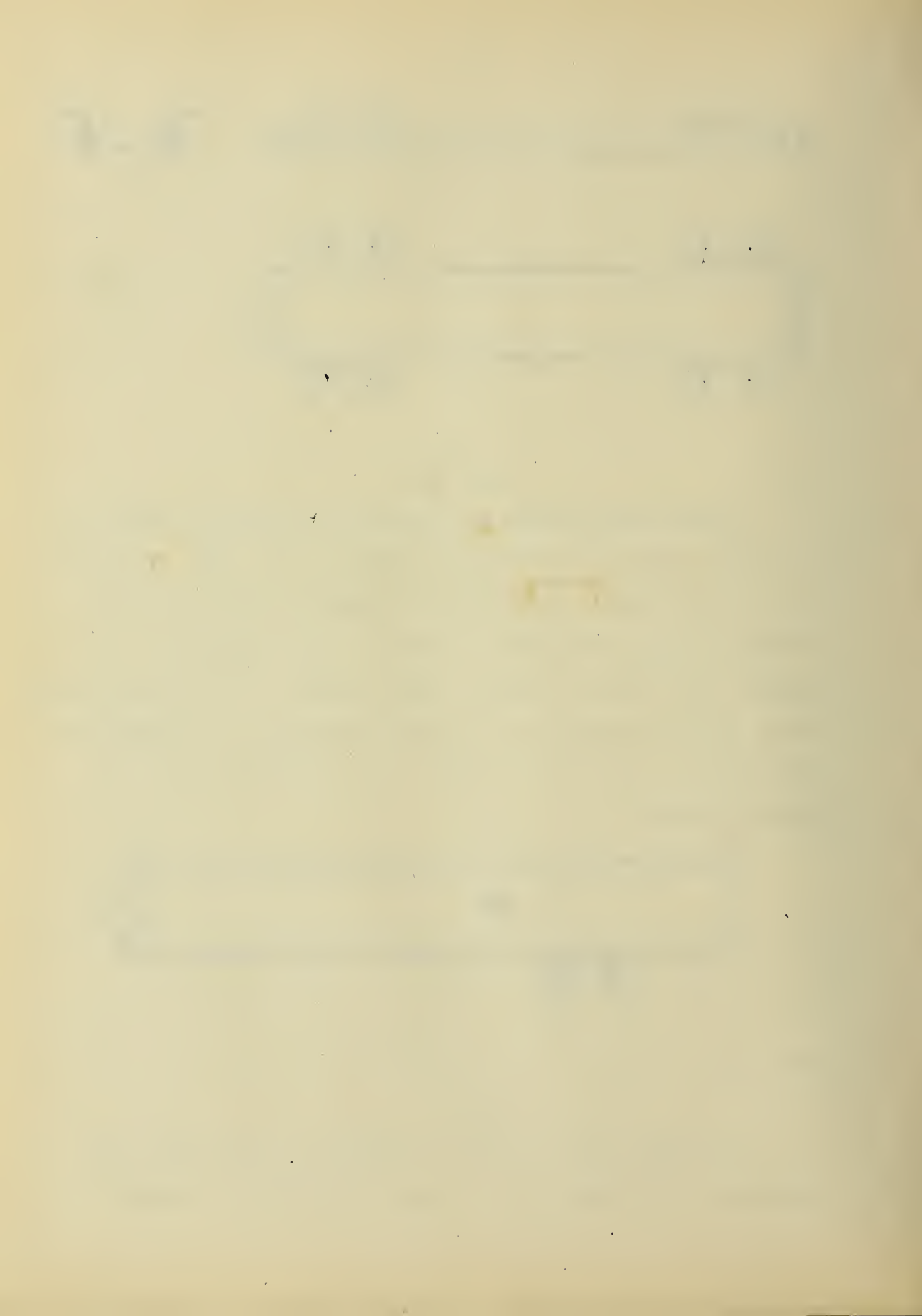


Fig. 83.

Fig. 83 shows a somewhat common but inefficient type of end strut, the shoe strut in this case being an eye-beam, the



joist raiser a channel. The form is not efficient on account of the method of transferring the stress from the strut to the masonry, the stress having to pass through the flanges of the eye-beam. The flanges of the eye-beam seldom exceeding  $3/8$  of an inch in thickness, but little chance is given the stress to distribute itself over the masonry, and as a result the masonry under the flanges wears away and tends to crack. This detail is not to be recommended in any case.

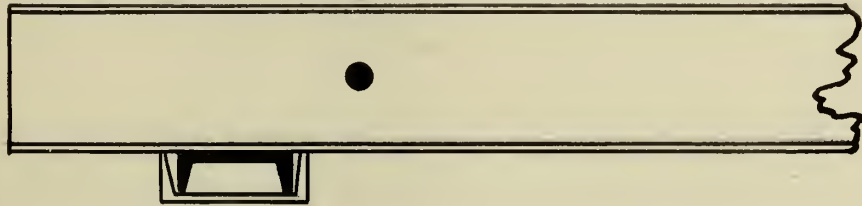


Fig. 84.

Fig. 84 shows a form of end strut similar in principle to that shown in Fig. 81 in which a channel rested upon a masonry plate. In this case, the joist-raiser channel rests upon a shoe-strut channel, the flanges of which tend to keep the joist raiser in place. The web of the shoe-strut channel distributes the stress uniformly from the flanges of the joist-raiser channel to the masonry, thus making the form similar to that shown in Fig. 81 as mentioned above. The form is very efficient in that the stress in the flooring is distributed very evenly to the strut and then in turn to the masonry even more uniformly on account of the increased area of contact offered. The form is also economical since only



two channels are required. Sufficient rigidity is obtained where the flanges of the upper channel fit snugly into the flanges of the lower channel.

The form is, therefore, to be recommended for bridges having panels of almost any length.

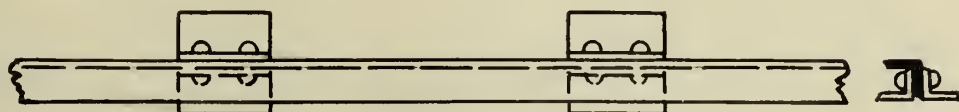


Fig. 85.

A very rare form of end strut employed on a pony truss is shown in Fig. 85. The shoe strut in this case, consists of a number of angles which are riveted to the joist raiser angles in the manner shown. The form is very efficient for a pony truss, in that it offers a uniform distribution of stress from the floor system to the strut and from the strut to the masonry. It is similar in principle to that shown in Fig. 80, an angle being used in this case instead of a channel. The form is economical in that only the standard sizes of angles are used. It is recommended for all pony trusses, but for bridges employing larger panel lengths, the form shown in Fig. 80 should be used instead.

Fig. 86 shows a somewhat rare form of end strut which is a modification of the form shown in Fig. 79. The eye beam rests upon small plates which in turn rest upon the masonry.



These plates are riveted to the flanges of the eye beam. The form is but little of an improvement over that shown in Fig.79 except that the little irregularities in the masonry do not need to be cut down to make the strut level, but the saving of expense in this way is compensated by the added expense of the plates. The form is, therefore, only to be recommended for bridges employing short panel lengths. If this type is desired for bridges with longer panel lengths, the plates should be made of corresponding larger area.

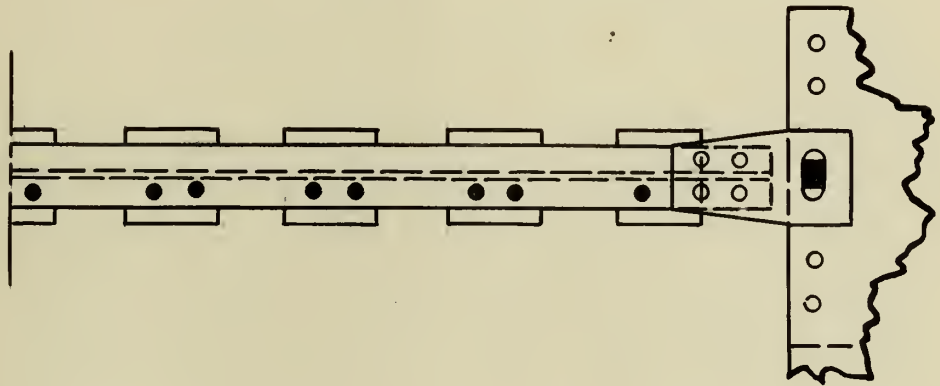


Fig. 86.

Fig. 87 also shows a rare form of end strut, this type being found on the 100-by 16-foot bridge built in Decatur Township by the Wrought Iron Bridge Co. This bridge has many unique features as is mentioned on p.31 . It might be said that the shoe strut in this case consists of an eye-beam supported on a number of small masonry plates. The joist raisers consist of a number a U-plates which hold the joists in place. The form is efficient, since the fastening of the



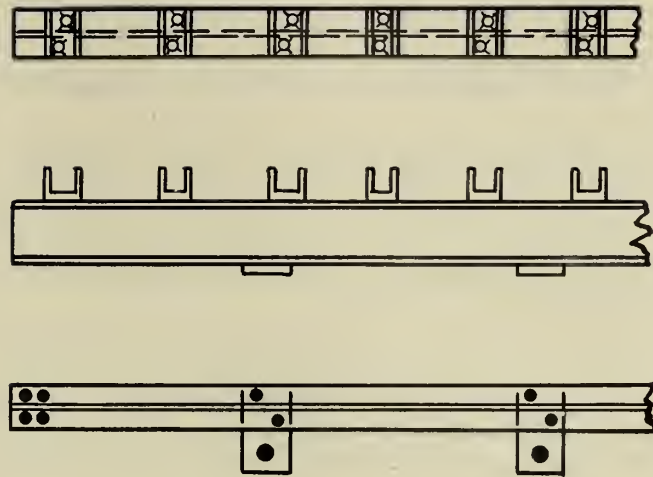


Fig. 87.

shoe plates to the masonry by bolts adds to the rigidity of the whole, although it also adds to the expense. The U-plates are used to keep the joists in place and, at the same time allow for the expansion due to temperature. The detail is, therefore, to be recommended for bridges with spans of almost any length but employing small panel lengths, these panel lengths not to exceed 20 feet.



ART. 14. PEDESTALS.

The pedestal is designed to transmit the stress from the end post and lower chord to the masonry on which it rests. Fixed pedestals may be employed for both ends of highway bridges of spans of less than 80 feet, Cooper's 1901 specifications stating that for all spans greater, roller nests should be provided at one end. For bridges of less span, smooth surfaces should be provided at one end for the free end to slide on. The pedestal should be very rigid to obtain the best transmission of stress. It should be also economical, and well protected from the elements.

Cooper's 1901 Specifications state, "All bed plates must be of such dimensions that the greatest pressure upon the pedestal stone shall not exceed two hundred and fifty pounds per square inch.

"Pedestals shall be made of riveted plates and angles. All bearing surfaces of the base plates and vertical webs must be planed. The vertical webs must be secured to the base by angles having two rows of rivets in the vertical legs. No base plate or web connecting angle shall be less in thickness than  $3/4$  inches. The vertical webs shall be of sufficient height, and must contain material and rivets enough to practically distribute the loads over the bearing or roller.

"Where the size of the pedestal permits, the vertical webs must be rigidly connected transversely. All the bed



plates under fixed and movable ends must be fox bolted to the masonry; for trusses these bolts must not be less than 1 1/4 inch in diameter.

"While the expansion ends of all trusses must be free to move longitudinally under changes of temperature, they shall be anchored against lifting or moving sideways."

Also it is the best practice to place the vertical connecting plates on the inside of the channels of the end posts

A good pedestal should also offer a good connection for the bottom lateral diagonal and the end strut. The fixed end, commonly called the pedestal, usually consist of two parts, the cast iron raiser, and the supporting part proper, which consists of riveted plates and angles. The cast raiser is necessary to raise the pedestal plate even with the top of the shoe strut. The pedestal plate is required to transmit uniformly the stress from the end post and lower chord to the cast iron raiser, which in turn transmits the stress to the masonry support. The pedestal plate furnish connections for the end strut. Cast iron raisers are now nearly all of standard form and dimensions. The only way in which cast raisers are found to vary is in the location of the holes for the anchor bolts, some standards having the bolt holes on the outside of the main body of the raiser while others have them on the inside. The method of placing the anchor bolt holes on the outside of the main body of the raiser gives the pedestal a more rigid hold on the masonry but is at the same time more expensive. The raiser should be securely fastened to the ped-



estal plate at the fixed end while at the free end slotted holes should be provided.

The pedestal plate proper usually consist of three plates and two angles as shown in Fig. 88. A rigid connection between plates and angles should be offered since all of the stress has to be transmitted through these to the raiser below. For this reason two rows of rivets are required in the vertical webs of the pedestal angles, as required by the specifications mentioned on p. 90.

Many forms of vertical connecting plates are used as will be seen by the following drawings. Their shape has no bearing upon the efficiency of the pedestal as long as they are within specifications, so the designer can use his judgment as to the most economical shape to employ. However, the form shown in Fig. 91 is recommended on account of the ease with which it may be cut.

Anchor bolts are of two types; those in which the end is forked and those in which the rod part is corrugated. The former are called fox-bolts. They are thought by many writers of specifications to be the more preferable since the forking of the end is thought to increase the efficiency a sufficient amount to warrant the increased cost. Anchor bolts are sometimes located at the middle of the sides of the pedestal plate while at other times they are placed in the opposite corners, the former method being preferred as all stress parallel to the plane of roadway is then transmitted evenly to the pedestal plate.



For all large bridges, pin plates should be fastened to the outside of the channels of the end posts to lessen the bearing and shearing stress on the pin.

Some of the commonest types of pedestals follow.

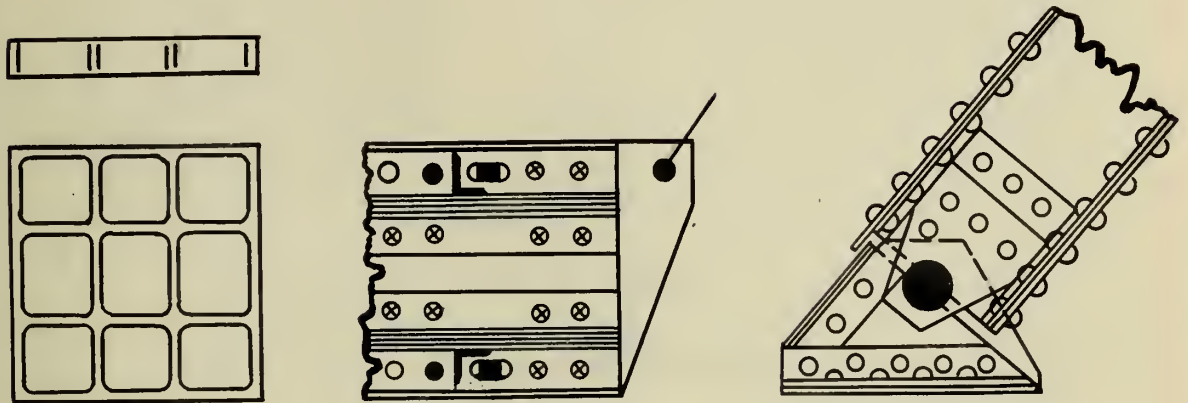


Fig. 88.

Fig. 88 shows a very common form of pedestal which consists of two parts, the cast raiser and the plate. The two parts are fastened together in this case, by means of anchor bolts. This detail does not offer a very good connection for the lower lateral diagonals since a forked rod is required, and usually employed and this entails extra expense. The form of connecting plate is good as it involves but little waste of material and is easily cut into that shape. It should, however, have two rows of rivets in the vertical legs of the angles as per the specifications mentioned on p. 90. The detail is recommended for spans of not more than 100 feet in length.



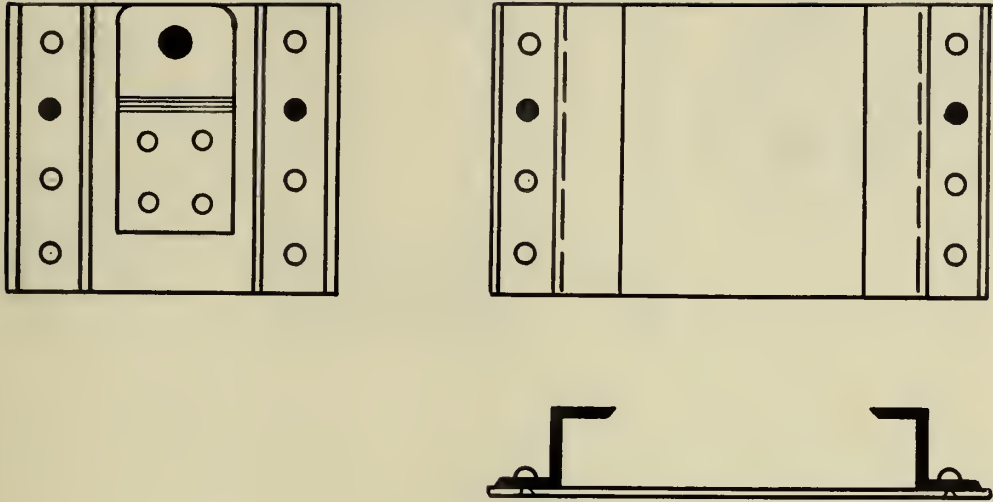
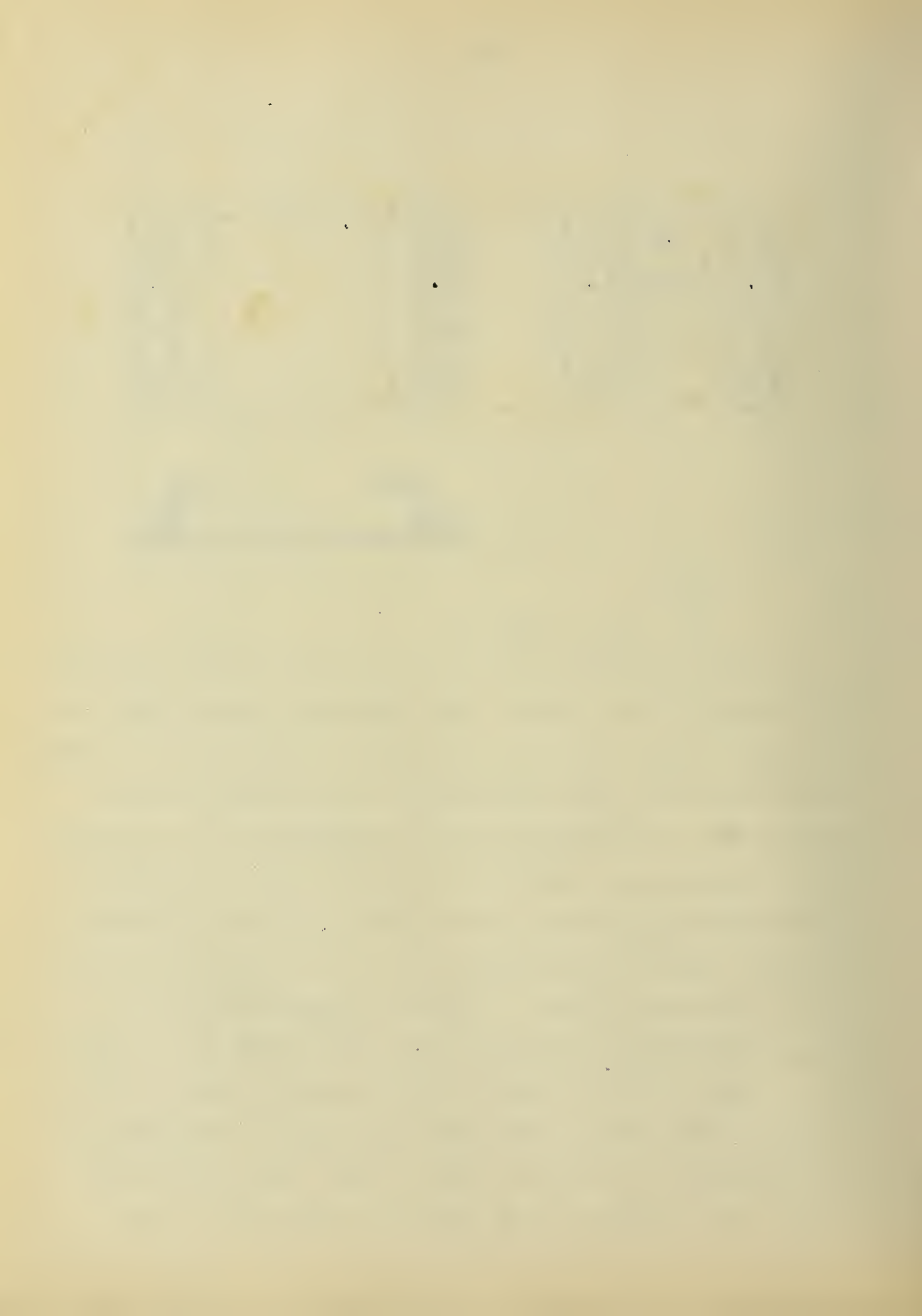


Fig. 89.

The detail shown in Fig. 89 is efficient and economical for bridges of small spans. The Z bars and masonry plate take the place of the cast iron raiser shown in Fig. 88 and are just as efficient and more economical for small spans, although the stress in the pedestal is not distributed quite so uniformly to the masonry. The bent plate at "a" takes the place of the extended plate shown in the preceding figure and offers a better method of connection to the lower lateral diagonals as is explained on p. 35. The form is, therefore, also to be recommended for bridges of spans under 100 feet in length.

Fig. 90 shows a form of pedestal that is not so efficient as that shown in the preceding figure. The connection of two angles in the manner shown is very inefficient, since very little rigidity can be obtained by one row of rivets



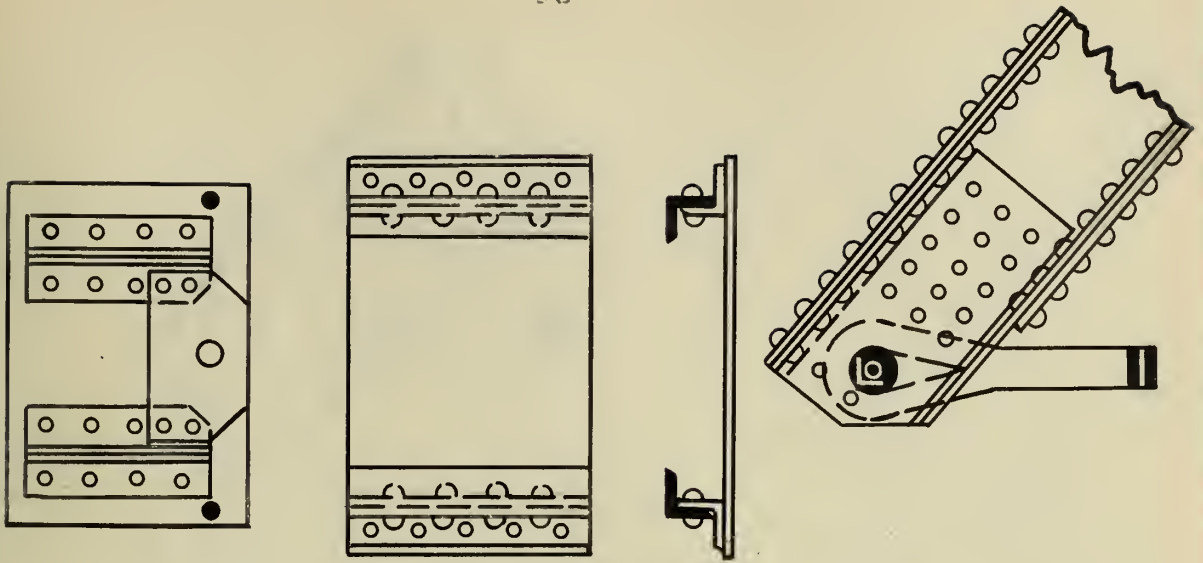


Fig. 90.

placed in the manner shown on account of the small area of contact offered. Although the connections offered, the shoe strut and lower lateral diagonals are good, the detail is, on account of the above fault, not to be recommended and if used at all should only be used on bridges of less than 60 feet span.

Fig. 91 shows a common detail for a pedestal in which no raiser except the masonry plate is employed. This form shows another economical method of cutting the connecting plate. The detail offers a good connection for both the lateral diagonals and the shoe strut. It offers a rigid connection to the pin in that there are two rows of rivets in the vertical legs of the angles. The pedestal plate is held in place by the two angles attached to the masonry plate. The form is, on account of its stability and efficiency to be



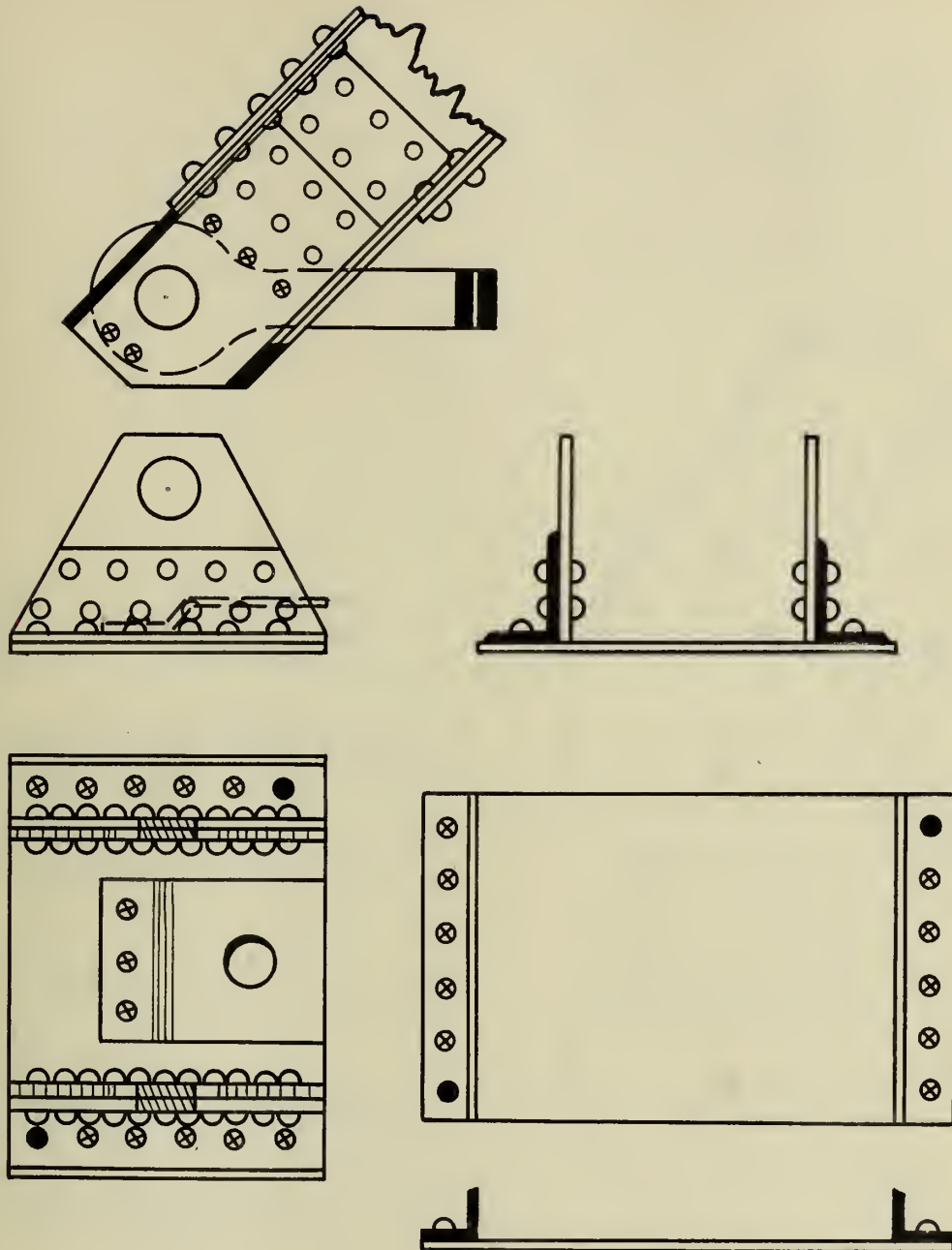


Fig. 91.

recommended for bridges of all spans.

Fig. 92 shows a pedestal very similar in principle to that shown in Fig. 88, the pedestal plate resting upon a cast iron raiser. The connecting plate instead of being fastened



only to the pin as is shown in Fig. 91, extends and is riveted far up on the inside of the channels of the end posts making the connecting very rigid. The ends of the end post channels should be planed to a smooth surface when cut in the manner shown in the above figure. The connection offered the lower

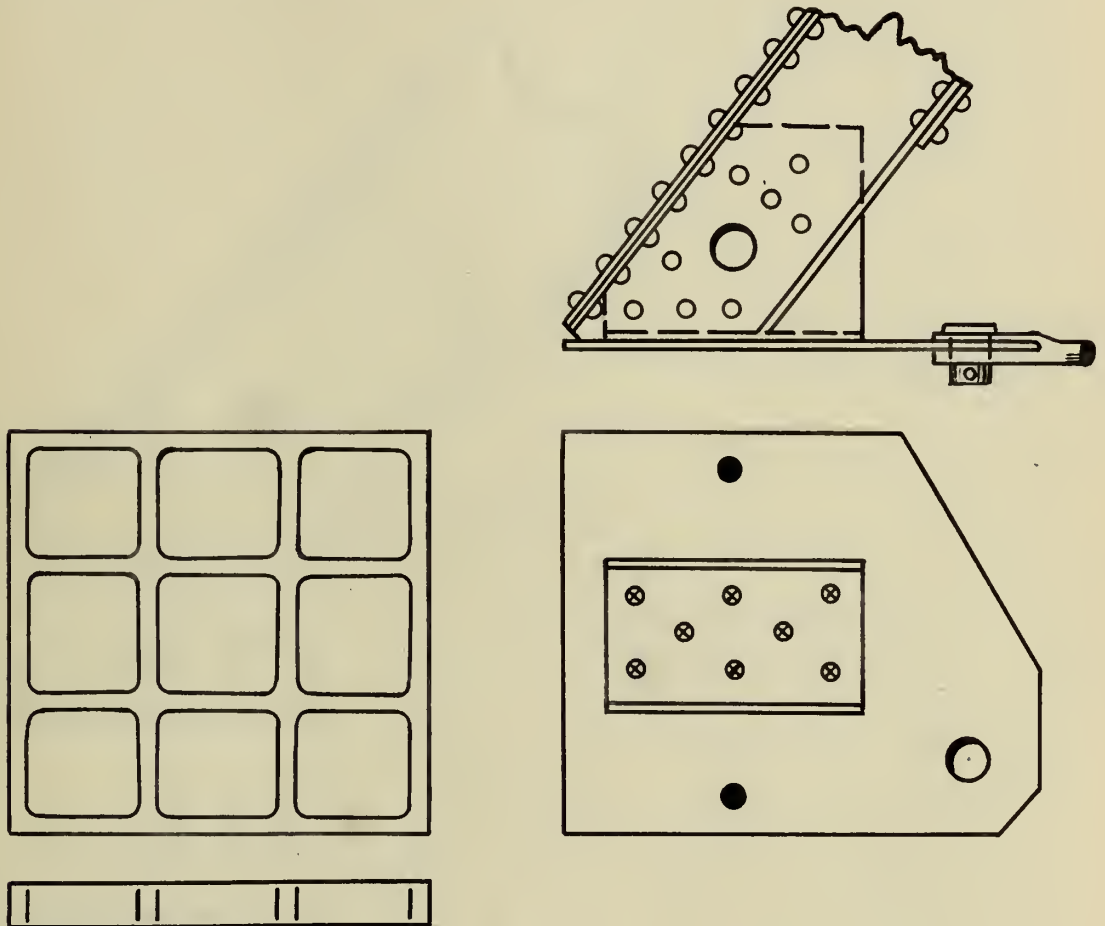


Fig. 92.

lateral diagonal and shoe strut is not good on account of the forked rod that is required and usually employed. Then too, the tilting effect caused by the wind produces tension in the rivets on that side on which the wind is acting. The form is, however, very simple and, therefore, economical. The use of the



form is allowable for all spans of less than 150 feet in length where the tilting effect is not too great to be successfully resisted by rivets in tension, but could be employed for larger spans if a better connection to the bottom lateral diagonal was offered.

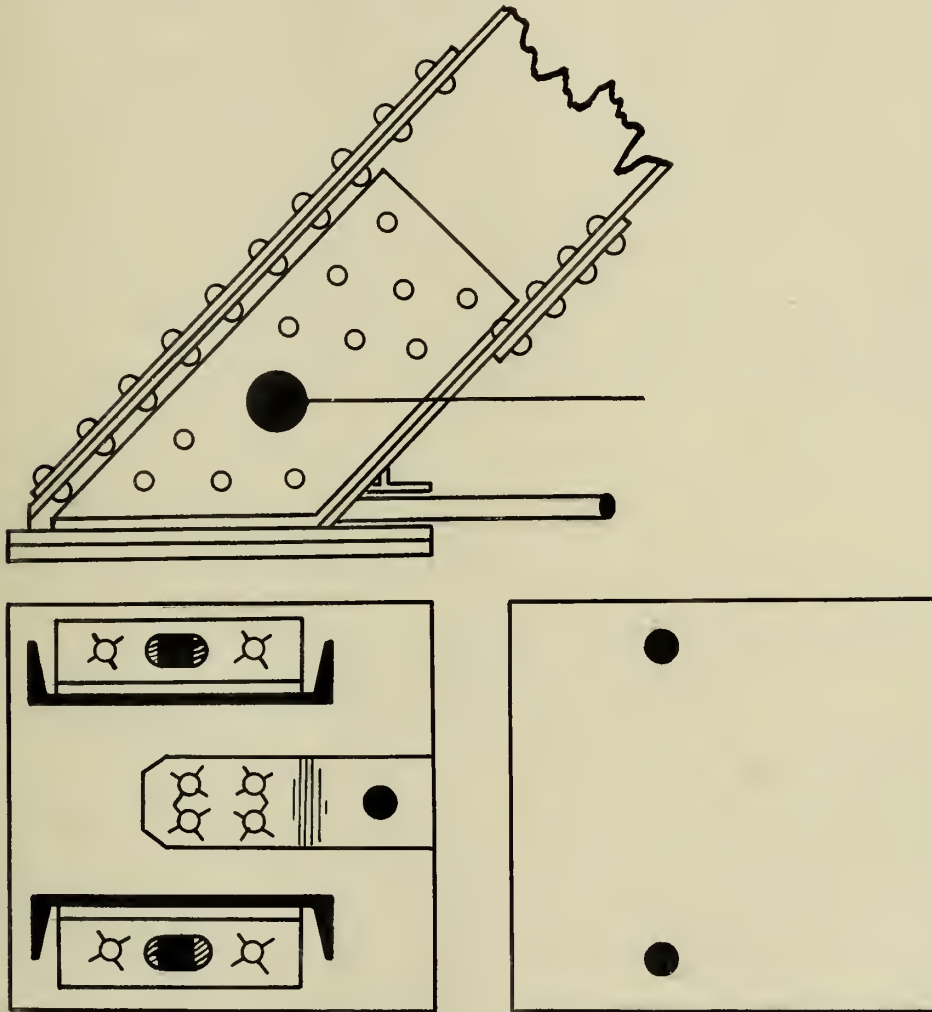


Fig. 93.

Fig. 93 also shows an efficient type of pedestal. It is built on the same principle as that shown in Fig. 91 the pedestal plate resting upon a masonry plate. There are, however, no angles fastened to the masonry plate to keep the ped-



estal plate in place but this is allowed for by the anchor bolts which keep both plates in place. The method of connection to the loweral lateral system and end strut is good. The cutting of the end post into the shape shown is somewhat expensive but little stress is thrown on the rivet and pin connections, and the connection can, therefore, be made much stiffer. When the end post channels are cut in the manner shown, they must have the end cut planed down to a smooth surface. The connecting angles, instead of only being fastened to the pin as is shown in Fig. 91 extend and are riveted far up on the inside of the end post channels, thus making the form very rigid. The form is, therefore, to be recommended for all spans.

#### ART. 15. ROLLERS AND MASONRY PLATES.

Coopers 1901 Specifications state:

"All bridges of over 80 feet span shall have hinge bolsters on both ends and at one end nests of turned friction rollers running between planed surfaces. These rollers shall not be less than  $2\frac{7}{8}$  inches in diameter for spans of 100 feet or less, and for greater spans, this diameter shall be increased in proportion of one inch for every 100 feet additional.

"The rollers shall be so proportioned that the pressure per linear inch of roller shall not exceed the product of the diameter in inches by three hundred pounds. (300 d)



"The rollers must be of machinery steel and the bearing plates of medium steel.

"The rollers and bearings must be so arranged that they can be readily cleaned and so that they will not hold water."

Roller nests should be placed on all bridges of over 80-foot span as per the preceding specifications. They are employed as a protection for the trusses against changes in temperature. The coefficient of linear expansion of steel is 0.0000065. Assuming one hundred degrees as the maximum change in temperature, and 80 feet as the length of the bridge, the change in length of the truss considered as a rigid member would be 6/10-inches. If this deformation would be prevented which would be the case if both ends were fixed, it is evident that in addition to the abnormal strains that would come on the pedestals, large strains would also come on the members of the trusses, and cause them to buckle. The reason that sliding nests instead of roller nests are not used is that the sliding surfaces very often become rusted while that end of the bridge is remaining stationary under a constant temperature, and then refuses to work when called upon. Sometimes dirt gets in between the friction plates and this has the same effect as rust. Roller nests on the other hand can stand much more dirt and rust before they refuse to work and are, therefore, used for large spans where expense permits.

Roller guides should always be provided to prevent rollers from rolling in any but the desired direction.

Roller nests should be bolted to the masonry as men-



tioned in specifications on p. 90.

In general, the same requisits of a good pedestal also applies to the roller nests, namely that it should be rigid, strong, economical, well protected from the elements, and should offer good connections for the end posts, bottom laterals, and the lower chords.

Masonry plates should always be at least  $3/8$ -inches thick and should not have a greater pressure upon the masonry than two hundred and fifty pounds per square inch. They should be bolted to the masonry in the same manner as the pedestals.

The various types of roller nests and masonry plates will now be discussed.

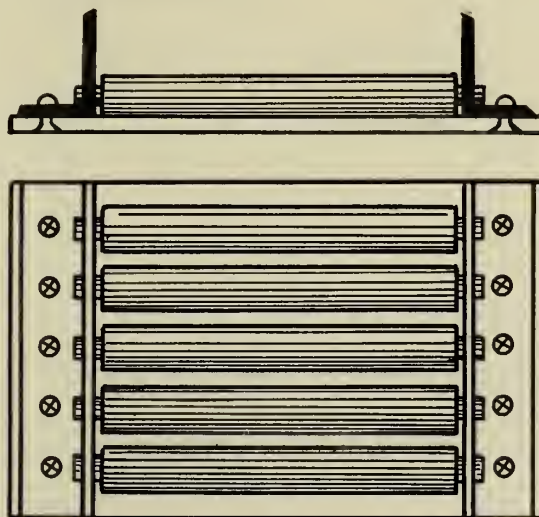


Fig. 94.

Fig. 94 shows the most common, although not the best type of roller nest employed on most highway bridges. It consists of a masonry plate to which are attached two angles



which serve as roller guides. Resting on this plate are the rollers, long cylinders of steel having axle ends to fit into the roller guide angles. Above the rollers is the pedestal plate and its connection to the end post. This plate may have any of the various forms shown in the preceding discussion on pedestals.

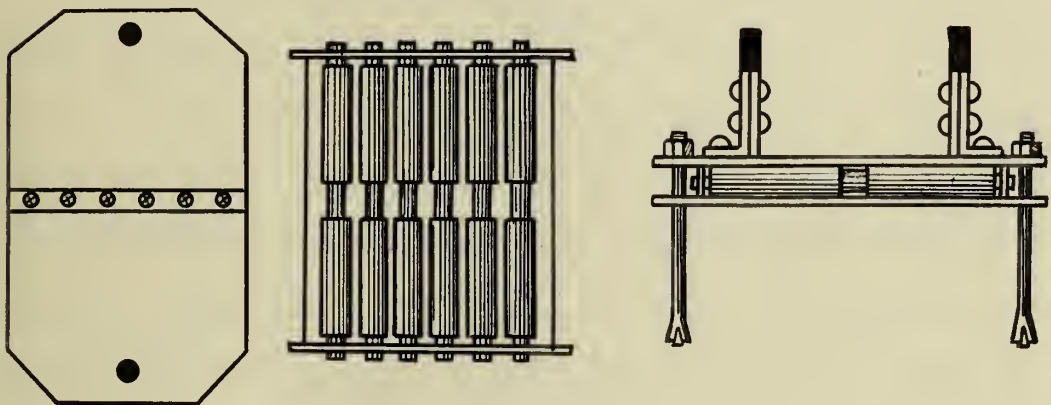


Fig. 95.

The detail is not as efficient as that shown in Fig. 95 where another roller guide is inserted in the form of a thin plate attached to the masonry plate by counter-sunk rivets, the rollers being cut down to a smaller diameter to allow the necessary space.

This detail is, however, more expensive than that shown in the preceding figure on account of the greater length required



of the rollers which must conform to specifications. This form is therefore only to be recommended for bridges of over 150-foot span, the preceding type being amply efficient for bridges of less span.

ART. 16. MINOR DETAILS.

(a) HUB GUARDS.

Hub guards usually consist of either angles or channels, angles being employed on bridges of short spans, and channels being used for the longer spans. They are used as a protection to the posts against injury from the hubs of the vehicles which pass over the bridge. Where there are no sidewalk supports, they serve as a fence to keep pedestrians from falling off the bridge. A good hub guard also gives an added rigidity to the bridge structure as a whole, since a post which is rigidly supported at three points is stronger than one supported at only two. While no stress is considered as obtaining in hub guards, they should nevertheless be of proportionate size according to the panel length of the bridge. A small angle serving as a hub guard on a long panel is unsightly and inefficient since it does not furnish any added rigidity to the post, and also because it cannot resist the dangers that arise in the case of greater and usually larger traffic.

The connection of the hub guard to the posts should be so as to prevent the hub guard from being easily wrenched loose



by traffic. It should be of such a section that a great amount of rigidity will be furnished the post from the hub guard.

There should always be at least two rows of angles or channels on each side of the bridge to serve as hub guards, whether a means of connection between the rows be offered or not. Hub guards are usually placed about three and one half feet above the floor. Lacing is often employed to advantage in hub guards since it increases the stiffness of the hub guard considerable.

Hub guards might be divided into two general classes, namely, those which offer attachment to posts where channel webs are parallel to roadway and those which furnish connections for posts having channel webs perpendicular to roadway. The former type will be taken up first.

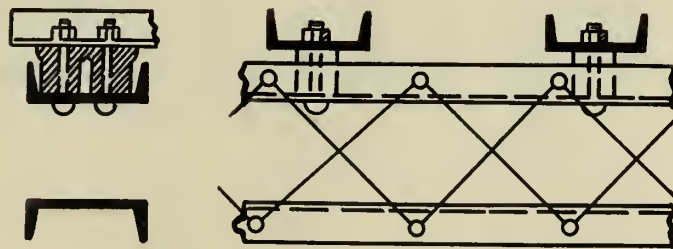


Fig. 96.

A very common hub guard and connection is seen in Fig. 96. Here the hub guard consists of two channels fastened by the backs of their webs to the posts by means of small circular shaped blocks as seen in the above figure. Connecting



bolts pass through the webs of the channels and the crown of the cast iron block or guard connection. The guard channels are made equal to the panel lengths. The detail is efficient for bridges of all ordinary spans since the blocks are not expensive and offer a fairly rigid connection to the posts. For bridges of very long spans, the form could be improved by the use of two small channels instead of the block. This would give a more rigid connection of guard to posts.

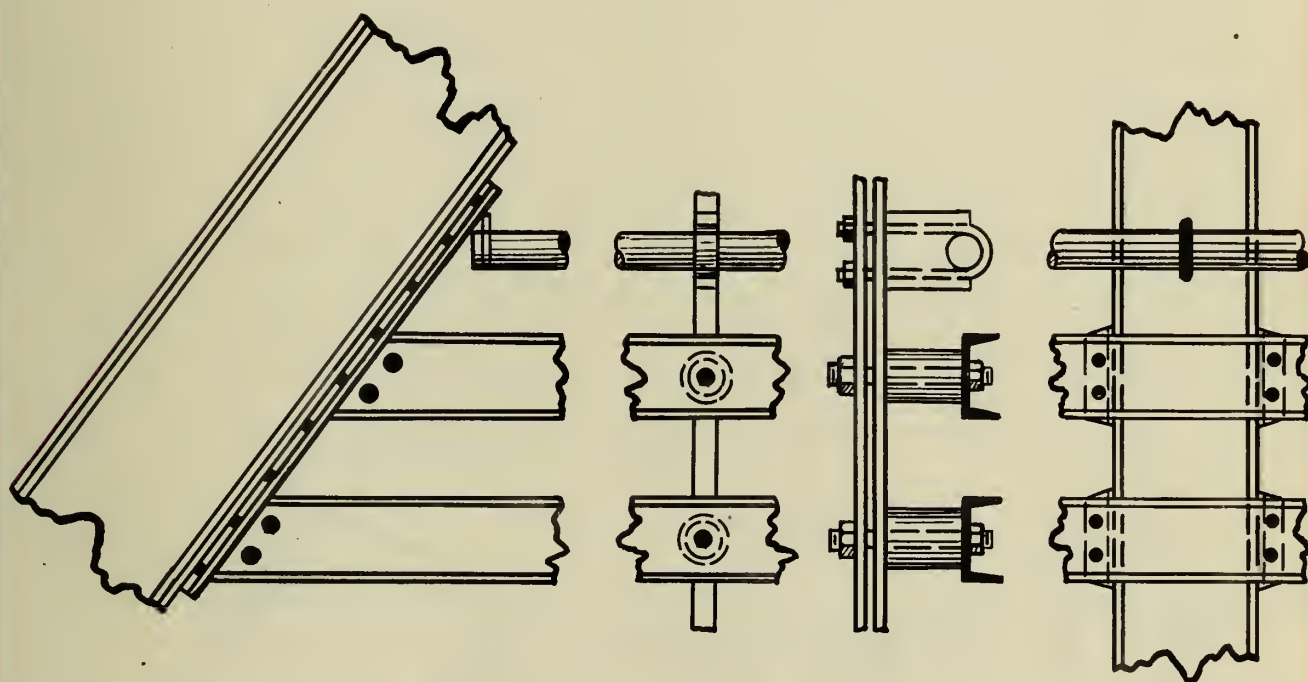


Fig. 97.

Fig. 97 shows the hub guard and connection employed in the Oakland Bridge which was built by the Chicago Bridge & Iron Co., in Coles and Douglas Co., and has a span of 180 feet. In this case two rows of channels and one row of gas pipes were used to form the hub guard, the connection to the



posts being made by combinations of angles, gas pipes, plates, and bolts as is to be seen in the preceding figure. The form is efficient since the gas pipes offer a sufficiently rigid connection of channels of guard to hip vertical on account of the great stability that the gas pipes have on the plane surface of the hip vertical bars and also because the connecting angles offer a rigid connection of guard to post channels through the flanges of the latter. A poor connection is offered the upper gas pipe forming the hub guard to the end post since only a very small pin is used for connecting purposes. Neither is the connection of the gas pipe to the intermediate posts very good on account of the hanger-like method of attachment to the same. The gas pipe is however probably not used for strictly hub guard purposes but for ornamental or hand-rail purposes instead since the channels are of sufficient size and have a sufficiently good connection to the posts.

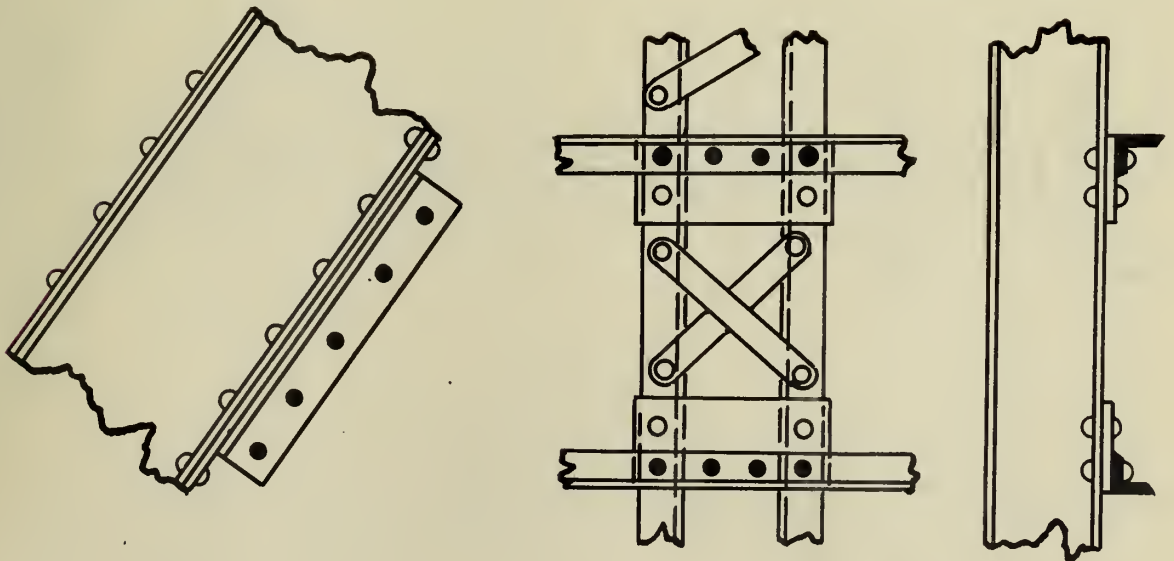
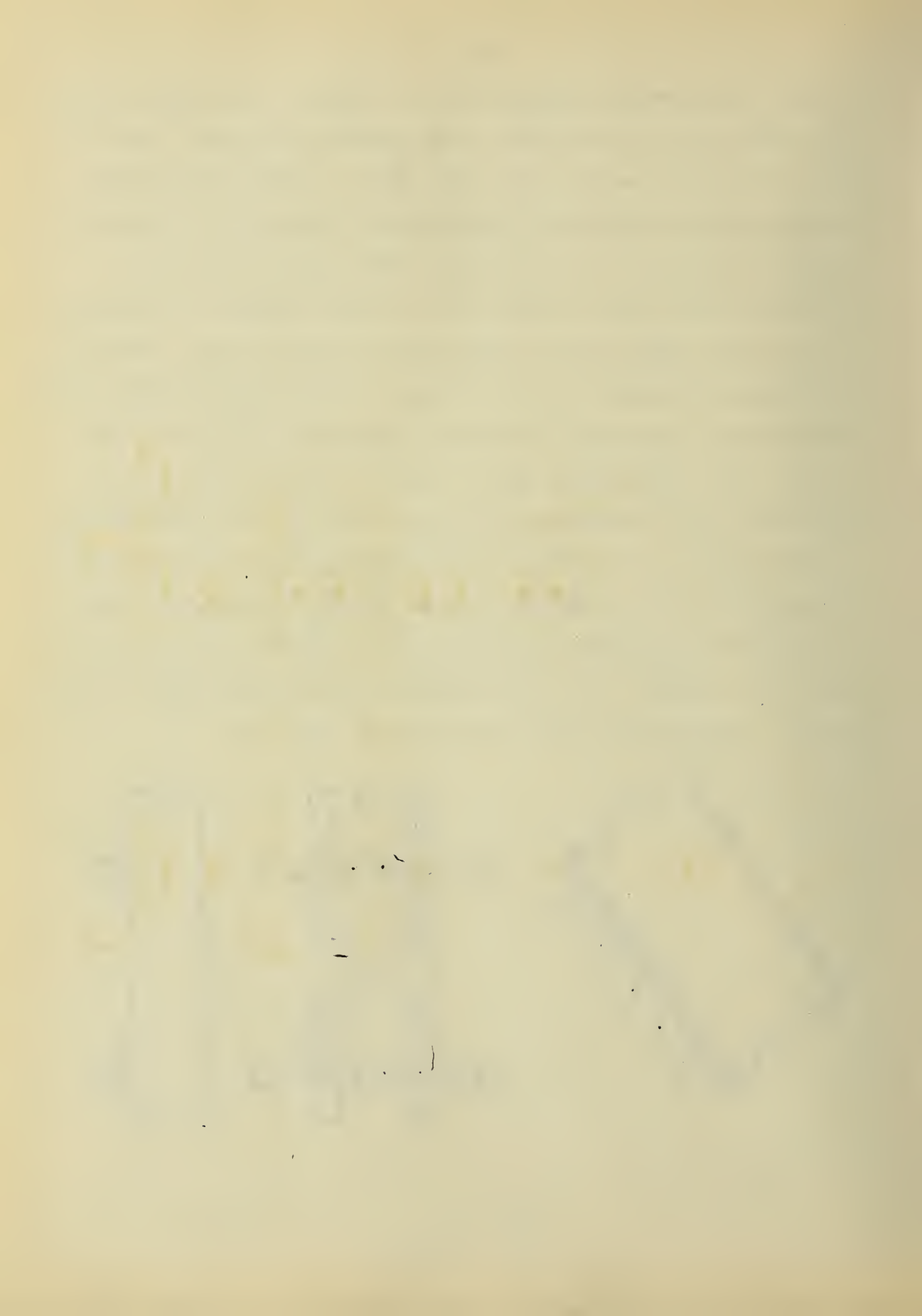


Fig. 98.



A somewhat common form of hub guard is shown in Fig. 98. It consists of two angles attached to the flanges of the channel posts by means of batten plates. The attachment to the end posts is offered by means of other angles. The detail is good since rigid connections are possible in most cases. The detail is however not to be recommended for panels exceeding 15 feet in length since it is too light. If however angles allowing two rows of rivets were used to form the hub guard, the form could be used for longer panels.

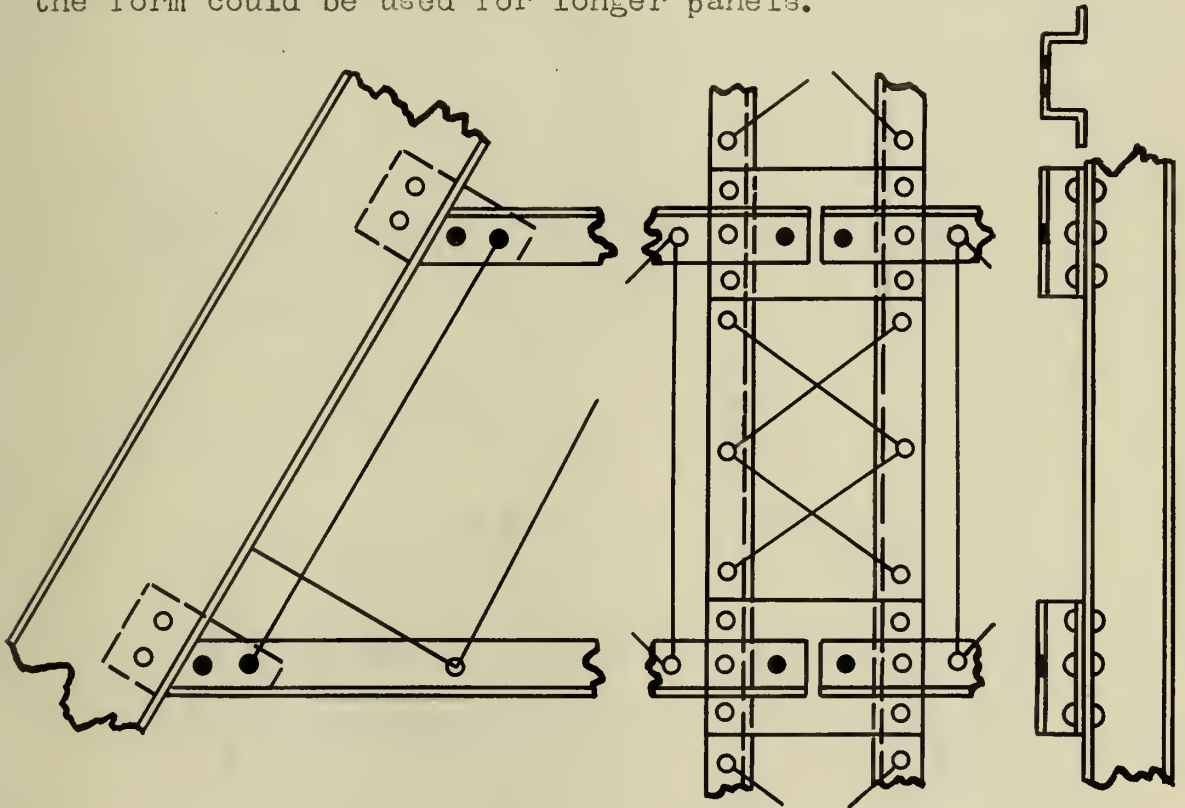


Fig. 99.

A common and efficient detail is to be seen in Fig. 99. It consists of two angles which are laced. The connecting to the posts is offered by means of a U-shaped plate which is attached by its flanges to the flanges of the posts-channel. The form is very good in that it is economical and



serves its purpose well since the guard is set well out from the post by means of the U-shaped plate. This lessens the dangers of vehicles striking the posts. The lacing of the angles adds greatly to the rigidity of the hub guard as a whole and consequently to the entire bridge structure. The attachment to the post is sufficient since eight rivets should hold any stress that will come in the hub guard. However, if greater strength were required, a batten plate might be fastened to the flanges of the post-channel and the plate flanges be made longer to admit four rows of rivets instead of two. The angles of the hub guard proper might then be made larger to admit two rows of rivets in order to furnish more rigid connection to the U-shaped plate. This improvement should only be placed on panels of over 25 feet in length, since it is too expensive for smaller panels. The detail is recommended for all panels up to 25 feet in length.

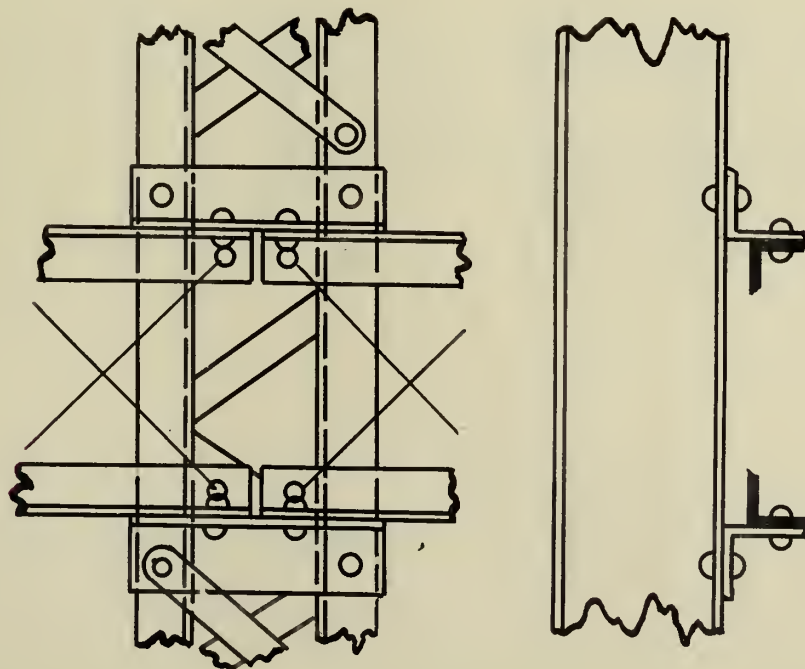


Fig. 100.



Fig. 100 shows an inefficient detail for a hub guard. It consists of two angles laced and attached to the post by means of clip angles. The lacing of the angles adds to the rigidity of the hub guard as a whole, and also offers a better protection for the posts against the traffic, but its efficiency is greatly reduced by the poor attachment offered the hub guard to the post. The form is allowable on spans having panel lengths not exceeding 15 feet, but for greater panel lengths angles allowing two rows of rivets should be used. A great part of the rigidity of the guard is then transferred to the post.

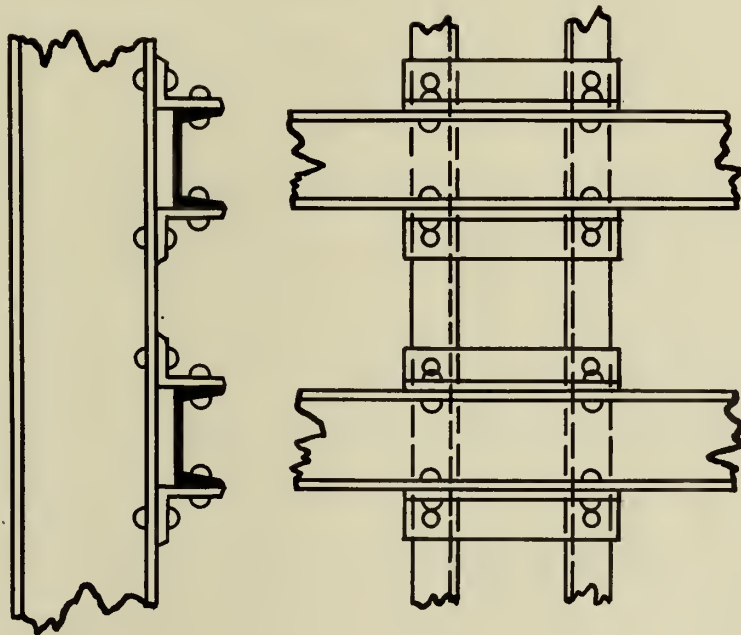


Fig. 101.

An efficient type of hub guard is shown in Fig. 101 which is but a modification of Fig. 98. It is, however, a decided improvement as a very efficient mode of connection to the post is offered in addition to the great strength and rig-



idity that obtains in the hub guard when laced. The detail is, when laced, recommended for bridges employing panel lengths of any length. Where channels are not laced, the form is recommended for bridges employing panel lengths not exceeding 20 feet in length.

(b) SIDEWALK SUPPORTS.

Sidewalk supports are usually only employed on the highway bridges of large cities, where the great amount of traffic forbids the use of the bridge floor for pedestrians. For highway bridges in the country, sidewalk supports are not necessary since the highway traffic is so light that ample opportunities are offered people to pass over the bridge in safety.

Sidewalk supports usually range from four to twelve feet in width, six feet being very common. Where sidewalk supports are used, a railing should also be used, to be placed at the outer extremity of the support to keep pedestrians from falling off the bridge. This railing should be at least four feet in height.

The flooring and joists on sidewalk supports are practically the same as that employed for the floor system proper. The floor beam however is seldom if ever made of an eye-beam since it is too expensive and would prevent a good connection being made to the posts. It is usually made up of a web plate and angles secured together by rivets in much the same way as in a plate girder. The web plate tapers towards the outside of the bridge where it has only the railing to support. The



web plate and angles act as a cantilever beam.

A common sidewalk support is seen in Fig. 102, which

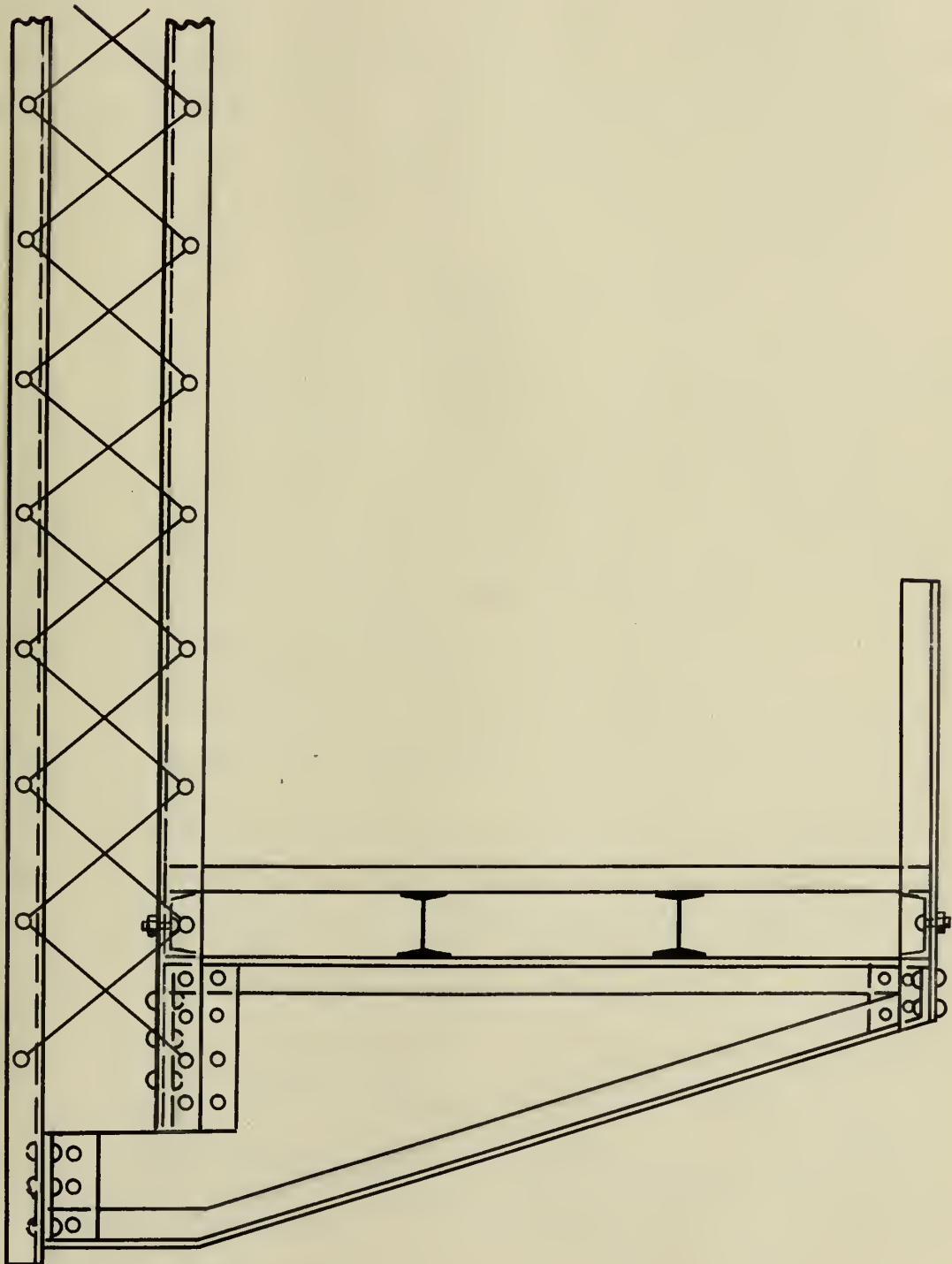


Fig. 102.



embodies all the features mentioned in the above discussion. The detail itself is very strong and rigid, but its attachment to the posts is weak, since the rivets in the upper part of the connecting angle are in tension when a load is placed near the outer extremity of the support. The form could be improved if a method of attachment to the railing were offered through the end post. The form is however efficient for bridges that do not take care of a large amount of pedestrian travel.

(c) FLOORS. (Wooden)

Wooden floors as employed in the majority of highway bridges consist of planks which are laid flat and in a direction perpendicular to the roadway, and about one fourth of an inch apart. The planks vary in width from six to twelve inches and from two and one half to three inches in depth, the most common dimensions being 8 in. x 2-1/2 in.

Another type of floor system is seen where there are two layers of planks laid diagonally across each other, and with the direction of the roadway. This is seen in large cities, where great strength in the floor system is needed. The type is poor on account of the poor ventilation offered the flooring and the fact that unless the flooring slopes towards the outside, and this is very seldom the case, water will soak into the planks and in time cause them to rot. Sometimes too, in large cities, Chicago being an example, wooden blocks are placed over a diagonal system of floor planks but the same result is sure to occur, i. e., the floor blocks will rot out in a short time due to the impossibility of properly draining



them. In such cases as in the above where a great deal of traffic has to be contended with, it would be much better to employ brick pavements, asphalt, or very thick planks to be laid in the manner described in the first paragraph of this discussion. The bricks to accomplish their purpose fully, should be cemented together in order to furnish a practically impervious surface. Concrete floors are also coming into prominence.

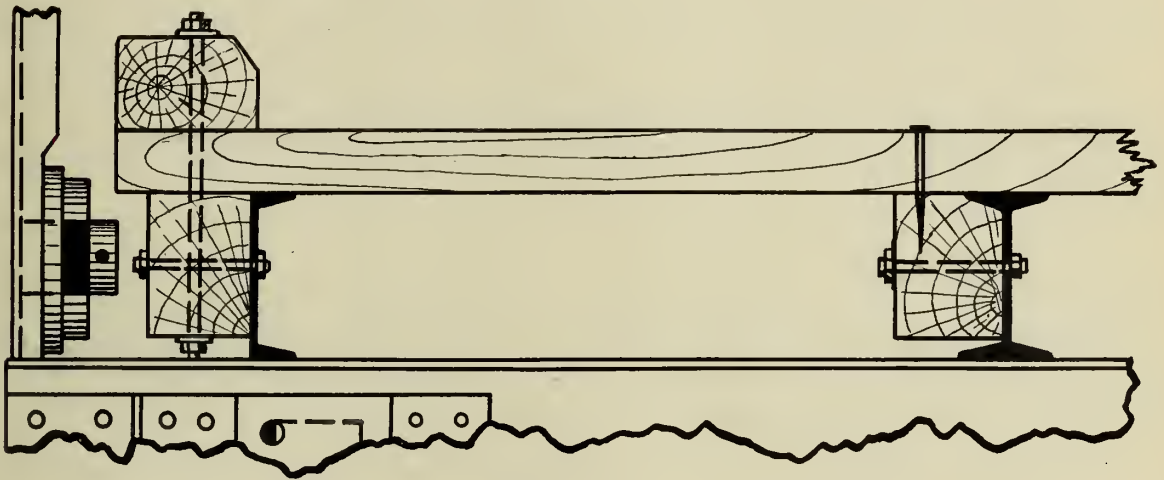


Fig. 103.

The planks of wooden flooring should be fastened to the joists in the manner shown in Fig. 103. The nailing planks are bolted to the eye-beams or channels whichever the case may be.

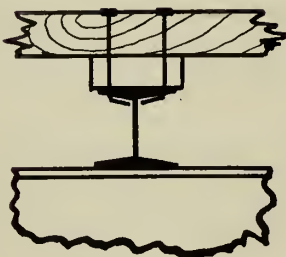


Fig. 104.

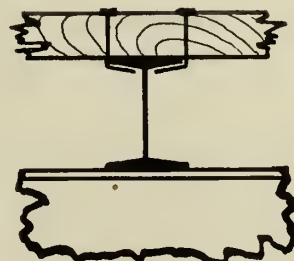


Fig. 105.



Another inefficient method is to be seen in Fig. 105, where the floor planks are fastened to the joists by the clinching of the spikes over the flanges of the eye-beams or joists. This is poor policy on account of the weak connection offered the joists, very little stress being required to partially bend the nail back to its original position and thereby leave the plank loose.

Felloe guards are usually placed at the outer ends of the planks on each side of the roadway at a distance of about two feet from the posts. They are usually of square cross sections varying from four to eight inches on a side. They should be made of the best oak and should be bolted to the nailing plank attached to the joist below. One edge of the plank is usually chipped off as is seen in Fig. 103. These felloe guards are placed in this position to protect the posts of the bridge from the traffic.

#### (c) FLOORS (Concrete)

While the use of concrete for various purposes has been making rapid advances in the recent years, it has not, however, made any marked progress in the way of being used for highway-bridge floor-purposes. This is in part due to the large cost of the concrete in the first place, and also to the general idea of the majority of highway commissioners that it increases the weight of the whole structure so much that a much larger and stronger bridge must be built to withstand the extra weight coming on the floor system. While it is true



that the first cost of reinforced concrete is quite large, it is also true that reinforced concrete is much stronger and better prepared to withstand those kind of loads which come on ordinary highway bridges. Also, the cost of maintenance is practically negligible, whereas for timber floors a constant replacing of worn out floors is necessary. The general idea of most highway commissioners, that reinforced concrete floors increase the weight of the whole bridge structure considerably is erroneous, for in the majority of cases wherein concrete flooring has been used instead of timber, the weight of the former has not been found to exceed that of the latter by more than ten percent. This is due principally to the fact that the use of joists, which constitute a large part of the weight of the entire bridge, is omitted. Then too, the use of concrete allows a better drainage system for the floor, and therefore better protects the floor beams from the weather and also makes a superior roadway for the traffic passing over it.

Therefore it is the opinion of the writer that the use of concrete floors for highway-bridge purposes is not as general as it should be, and that it is false economy to use timber floors which require constant repairing, instead of concrete which requires practically no repairs and will last as long as the bridge structure itself; this being especially true for bridges of spans of less than 80 feet in length. It would be true also for larger spans where it not for the fact that the highway commissioners in the majority of counties have



very limited funds with which to work, thus necessitating the building of somewhat flimsey structures.

Where reinforced concrete floors are used, the workmanship and material employed should be of the very best since poor work of this kind is practically worthless.

Concrete floors should always be at least six inches thick at the thinnest portion, should also have a layer of neat cement at least one inch thick to serve as a crown and as a protection to the concrete below, should in all cases be covered with gravel or some other common road material for protection against the abrasion of the horses hoofs and should always be furnished with sufficient crown at the middle of the roadway to furnish good drainage to the curbs. The curbs should also be set to a grade to furnish good drainage for the floor system longitudinally.

There are many methods employed for reinforcing concrete in bridge floors, two of the more common of which will be shown.

A very common and efficient method of reinforcing concrete floors in highway bridges are shown in Fig. 106 which represents the type of floor employed in the Maumee River Bridge of 166-foot span, and which was built by the Massillion Bridge Co., at Waterville Ohio. The weight of the joists is not done away with in this case as the joists serve as the reinforcement of the concrete while the added weight of the concrete floor in this case become quite large; the floor system as a whole is made very strong. The tying of the joists together



by the rods, also makes it very rigid and serves as a protection to the concrete against vibrations of the floor beams. The cost of the corrugated arches is not great, and the form may therefore be said to be economical for bridges of over 150-foot span where the added weight of the floor system, due to the concrete is not great as compared with the entire weight of the bridge.

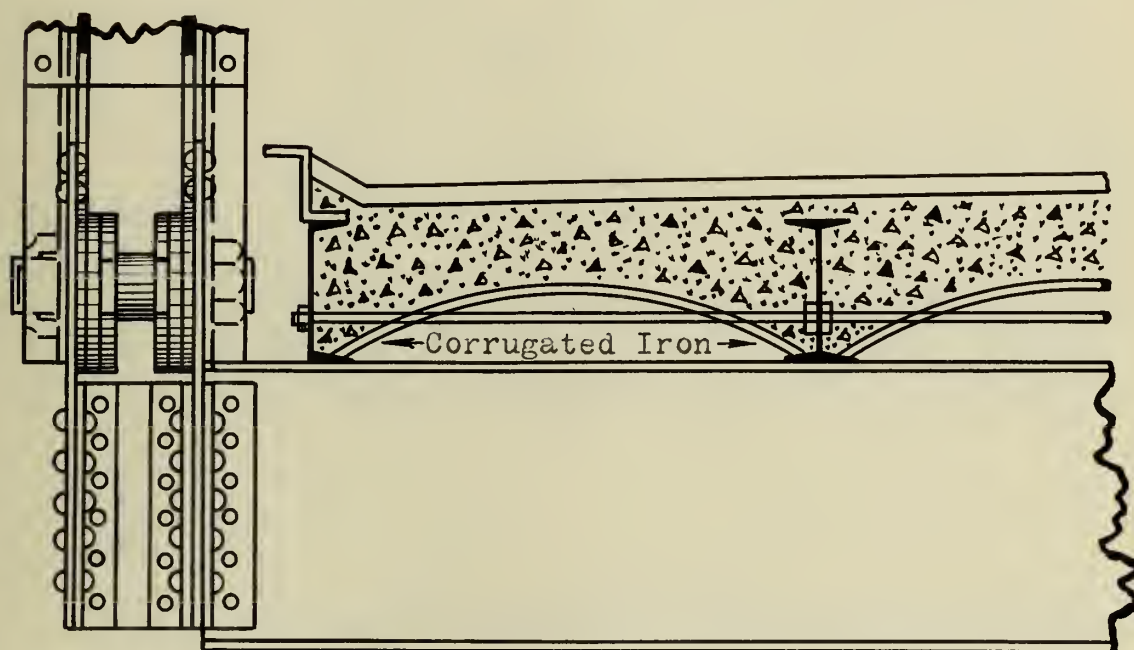


Fig. 106.

Fig. 107 shows a type of reinforced concrete construction that is common to smaller highway bridges. The form is very strong since it has two layers of reinforcing bars in the concrete, which are laid in a perpendicular direction with each other. The form is economical since no steel joists are required as in the preceding form, thereby reducing to a great extent the added weight due to the use of the concrete flooring.



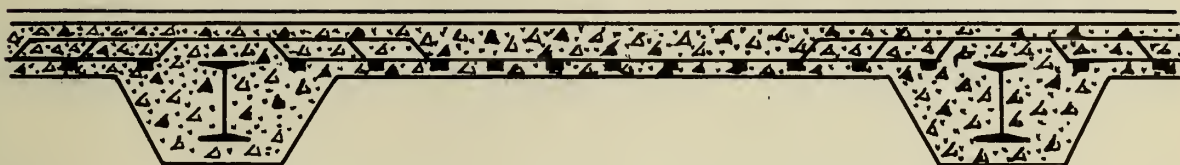


Fig. 107.

This form should not, however be, used on bridges employing a greater panel length than 16 feet, as too large a strain will then come upon both the concrete and the metal arch in the middle of the panel length. The form is to be recommended for small spans of panel lengths no greater than 16 feet.

#### ART. 17. CONCLUSION.

The constant use of certain forms of details for the various parts of highway bridges is not to be recommended on account of the varying conditions that are to be met with, and the fact that no progress would be made in design if various designs of details were not used experimentally. Therefore it is not the purpose of the writer to state specifically the only forms that are ever to be used, but rather to suggest the details that, in the majority of cases, are to be recommended under normal conditions.

The essential features that are required of the details



which are recommended are in general, economy, efficiency, simplicity of construction and erection, and rigidity and strength. A summary of the most efficient type follows, the various details being taken up in the same order as discussed in the preceding articles. No attempt will be made in the discussion of the relative merits of each detail as this is considered to have been treated substantially in the preceding articles. Ready reference to discussions may however be made by means of the figures.

END POSTS AND UPPER CHORDS:- Two channels, lacings and a cover plate arranged as shown in Fig. 1, p. 2 are considered as forming the best detail for end post and upper chord construction. The end post and upper chord sections should be made of the same dimensions. They should be designed to comply with the requirements mentioned on pages three and four.

INTERMEDIATE POSTS;- The use of intermediate posts with their channel webs parallel to the roadway with the connections shown in Fig. 3 p. 9, to the upper chord is advised for bridges of spans up to 130 feet in length. For greater spans, the use of intermediate posts with channel webs perpendicular to roadway with connection to upper chord as shown in Fig. 4, p. 9 is recommended.

HIP VERTICAL;- The hip verticals shown in Fig. 7 and 8, p. 11 represent the most advisable types for bridges with intermediate posts having channel webs parallel and perpendicular to roadway respectively. For very small spans the form



shown in Fig. 11 p. 14 is suggested.

MAIN AND COUNTER TIES:- Main ties, in pin and connected trusses, should always consist of eye-bars which should be packed well and be placed as near each other on the pins as practical. The ratio of the width to the depth of the bar should be as explained on p. 16. Two bars should always be used for counter purposes as shown in Fig. 17, p. 20, turn buckles being provided for each in all cases. However, wherever only one bar is used it should be forked as shown in Fig. 15, p. 18, or less preferably placed adjacent to the other ties and eccentrically on the pin as shown in Fig. 13, p. 18. For riveted trusses, the form shown in Fig. 16, p. 19 is recommended.

LOWER CHORDS;- For all pin connected trusses the lower chord should be made up of eye-bars employing a ratio of width to depth as mentioned in preceding paragraph. Wherever a number of tension members are acting about a pin, they should be alternately placed on each side of the pin to avoid a large bending moment. The chord members nearest the middle of the truss should be placed on the outside as explained on p. 21 and shown in Fig. 20, p. 22. For riveted trusses, the most efficient form is shown in Fig. 19, p. 22. Fig. 19 and 20, p. 22 show the most common location of the post for the two types respectively and also the method of packing employed in each case.

TOP LATERAL STRUTS;- Fig. 23, p. 24, shows an economical and efficient form of top lateral strut for bridges of spans up to 100 feet in length. An efficient and top lateral



strut is represented in Fig. 22 p. 23. It is also recommended for spans not exceeding 100 feet in length. For greater spans the form shown in Fig. 28, p. 26, or Fig. 29, p. 26 are recommended as they are far in advance of any of the other types in effectiveness.

TOP LATERAL DIAGONAL AND CONNECTIONS:- The use of angles for top lateral diagonal purposes is to be preferred for reasons mentioned on page 28. Where rods are used the connections shown in Fig. 31 and 32 p. 30 are to be recommended for reasons mentioned on that page.

BOTTOM LATERAL DIAGONALS AND CONNECTIONS:- Bottom lateral diagonals should be designed as described on p. 33. Rods in this case are to be preferred over angles owing to the limited available space that is allowed for connection purposes. Fig. 39, p. 35 represents the best method of connection of diagonal to pedestal for reasons stated on that page, while Fig. 43 is the best for connection of diagonal to intermediate floor beams.

FLOOR BEAM CONNECTIONS:- There are two types of floor beam connections namely, those offering connection of floor beam to posts whose channel webs are perpendicular or parallel to roadway respectively. Floor beam connections are usually made below the chord pin, but it is better to make the connection above for reasons explained on p. 61. Since floor beam connections below the chord pins are employed by so many designers, however, it might be well to suggest a few commendable types. Fig. 62, p. 62 shows a commendable form of floor beam



connection where channel webs of posts are perpendicular to roadway and connection is made below the pin. This detail should however be only used for spans employing panel lengths of not more than 18 feet. The details shown in Fig. 62 p. 62 and Fig. 63 p. 64 are recommended for bridges employing panels of any practical length.

Where the floor beam connection is located above the chord pin and channel webs are perpendicular to roadway, commendable types are to be seen in Fig. 71 p. 73, Fig. 72 p. 74 and Fig. 75, p. 77. Fig. 66 and 67 are recommended for bridges employing panel lengths not exceeding 20 feet while Fig. 77 p. 79 is suggested for the larger panel lengths. The best detail for floor beam connection purposes where connection is made above the chord pin and posts, have channels with their webs parallel to the roadway is seen in Fig. 74 p. 76 for reasons mentioned on that page.

PORTALS AND SWAY BRACINGS;- A very desirable portal should comply with the requirements mentioned on pages 41-45. The types shown in Fig. 48, 49, 52 and 53 are to be recommended for reasons mentioned on p. 47-51. Fig. 52 p. 50 shows a very rigid form of portal, and especially to be recommended for spans greater than 150 feet. A very desirable type of portal also is shown in Fig. 56, p. 53. The detail is also recommended for spans greater than 150 feet in length.

SWAY BRACING:- The form of sway bracing shown in Fig. 59, p. 57, improved as suggested on that page, is recommended for bridges of long spans employing trusses of great depth,



for reasons explained on that page. The detail shown in Fig. 58 p. 56 is also recommended for the same reasons. It might be said to be preferable to that shown in the above mentioned figure on account of its being the less expensive.

**JOISTS AND SHOE STRUTS:-** Joists should be arranged as shown in Fig. 78 p.81 and should comply with the requirements mentioned on p. 80. For bridges of panel lengths not exceeding 14 feet, the end strut shown in Fig. 79, p. 82 is recommended. For larger panel lengths, details shown in Fig. 80 p. 83 Fig. 81 p. 84, Fig. 82 p. 85 and Fig. 84 p. 86 are recommended, the form illustrated in Fig. 84 being especially commendable. For the roller end, the form shown in Fig. 87 can be used. For pony trusses employing short panel lengths, the detail shown in Fig. 85 p. 87 is recommended for reason mentioned on that page.

**PEDESTALS:-** Pedestals should comply with the specifications mentioned on pages 90 and 91. Pedestals represented in Fig. 88 p. 93 and Fig. 91 p. 96 are recommended for spans of any practical length. The form shown in Fig. 93 is recommended for bridges of spans exceeding 100 feet in length.

**ROLLERS AND MASONRY PLATES:-** Rollers and masonry plates should comply with specifications mentioned on pages 97 and 100. The roller nest illustrated in Fig. 94 p. 101 is recommended for reasons mentioned on p. 102, but the detail shown in Fig. 95 p. 102 is allowable on all ordinary spans.

**MINOR DETAILS:-** (a) **HUB GUARDS:-** Hub guards should be built with the aims in view mentioned on p. 103. For



bridges employing ordinary panel lengths, the form shown in Fig. 97 p. 108 and Fig. 99 p. 107 are recommended. For longer panel lengths, form shown in Fig. 100 p. 101 and Fig. 99 improved as suggested on p. 108 are recommended for reasons mentioned on those pages.

(b) SIDEWALK SUPPORTS:- For ordinary pedestrian travel, the sidewalk support shown in Fig. 102 p. 111 and discussed on p. 110 is recommended. For excessive travel, notches might be cut in the channel webs and the angles placed in position from the inner side, and the connection to the post be made by field riveting. This would prevent tension in rivets and would make the support much safer and stronger.

(c) WOODEN FLOORS:- Wooden floors should be made of the best oak planks of widths varying from 6 to 10 inches and depths from 2-1/2 to 3 inches. They should be laid as stated in the first paragraph of p. 112. The planks should be attached to the joists as shown in Fig. 103 p. 113, and felloe guards as shown in that figure should always be used.

(c) CONCRETE FLOORS:- Concrete floors should always be laid according to some set of first class specifications. The concrete should always be of first class quality and reinforcement should always be present and in ample quantities. For the shorter spans employing short panel lengths not exceeding 16 feet, the form of reinforced concrete flooring shown in Fig. 107 p. 118 is recommended, while for larger spans, employing larger panel lengths, the form shown in Fig. 106 p. 117 is suggested for reasons stated on that page.

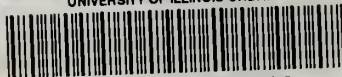


The various details that are to be recommended have been summarized in the above, and suggestions made as to the most appropriate use of each. It is now left for the designer to use his judgment.





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